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PRESTRESSED CONCRETE INSTITUTE

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Journal of the prestressed concrete institute

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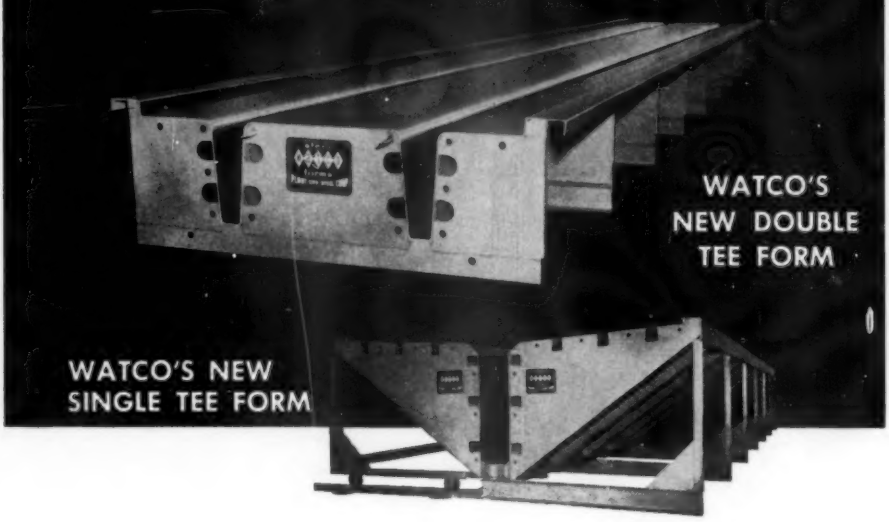

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June, 1961
Vol. 6, No. 2

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

The **JOURNAL** is published quarterly by the **PRESTRESSED CONCRETE INSTITUTE.**

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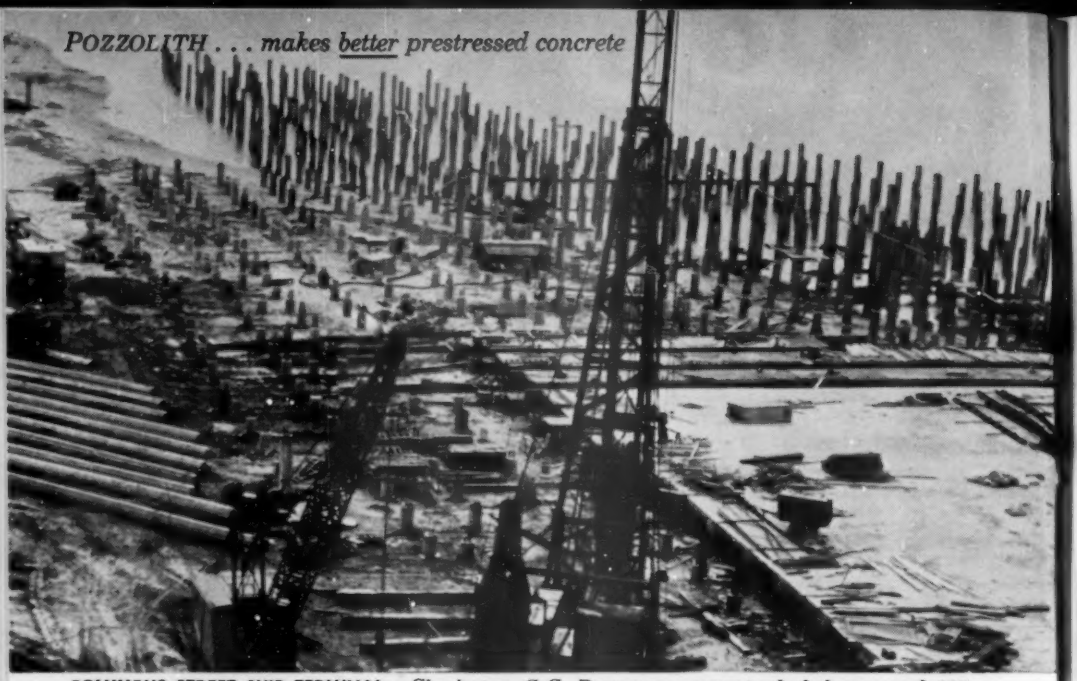

COLUMBUS STREET SHIP TERMINAL—Charleston, S.C. POZZOLITH concrete deck beams and 10″ concrete slab (lower right) are used on top of the pilings for 750,000 sq. ft. dock. Construction Agency: South Carolina State Ports Authority, Charleston • Architect-Engineer: Lockwood-Greene Engineers, Spartansburg, S. C. • General Contractor: J. A. Jones Construction Co., Charlotte • Prestressed Concrete and POZZOLITH Ready-Mix Concrete Supplier: Concrete Materials, Inc., Charlotte.

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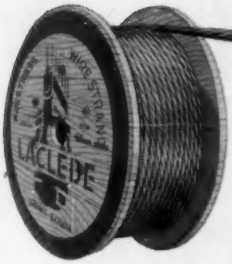

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Prestressed concrete piles and deck units were used in the construction of a new Atlantic Coast Line span in South Carolina. On this project, one of the first major prestressed concrete railroad trestles in the U. S., it was estimated that over $60,000 was saved by choosing the prestressed units over conventional concrete construction.

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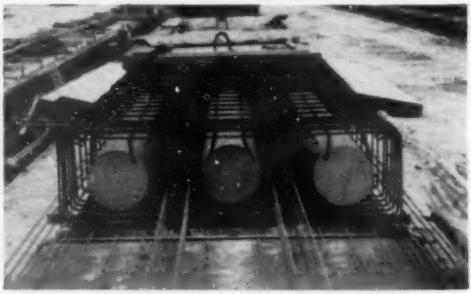

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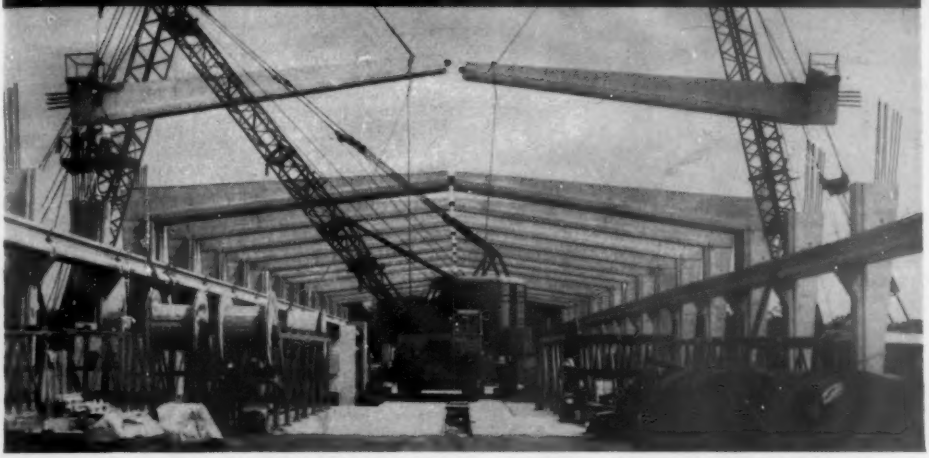

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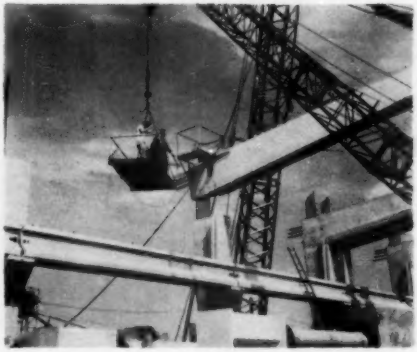

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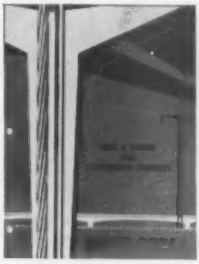

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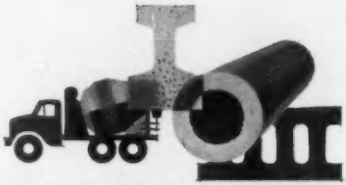

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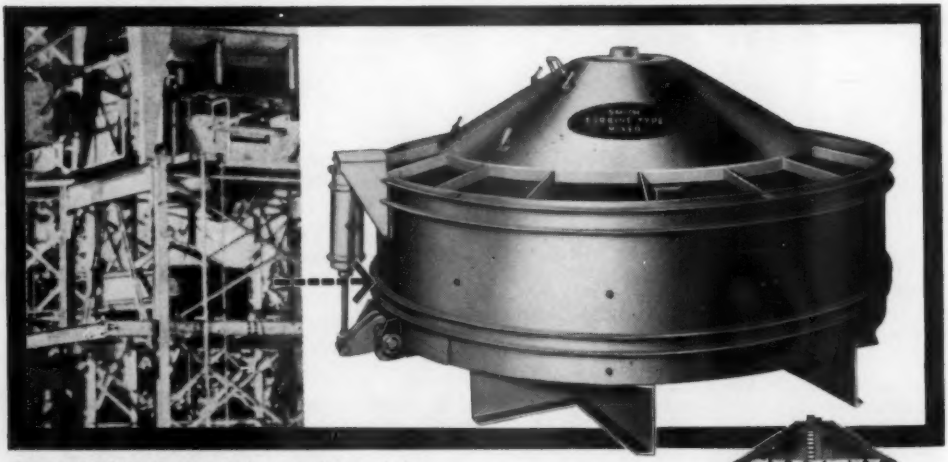

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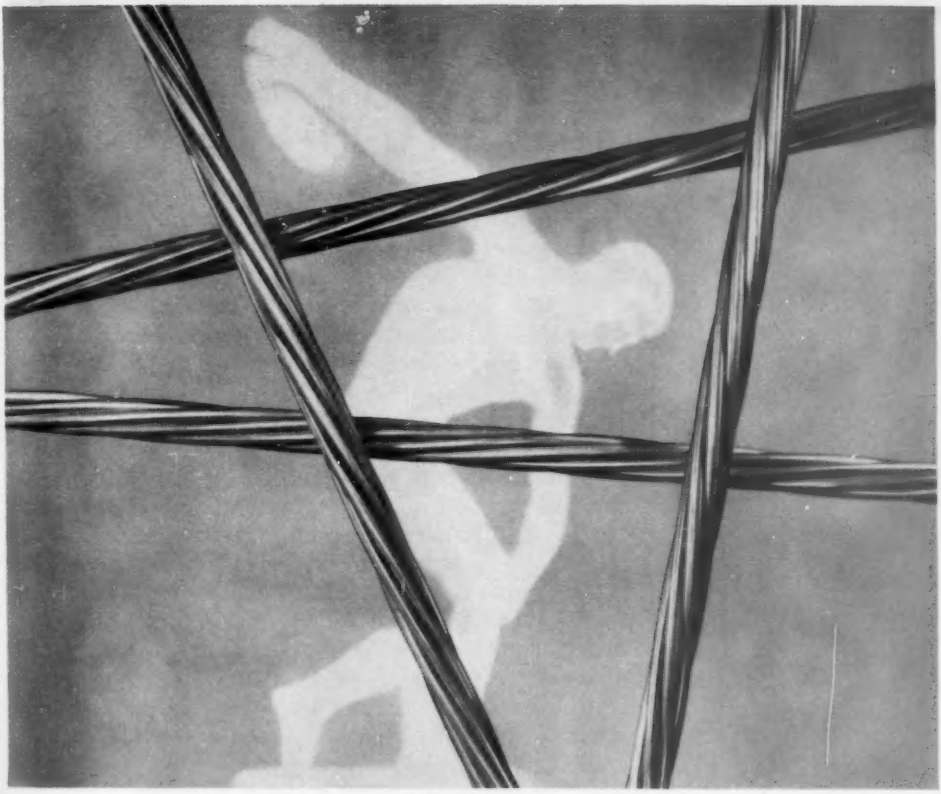

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By Charles Luckman, A.I.A.
President,
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GUEST EDITORIAL

We now have in this country investments in structures of all kinds amounting to more than $850 billion, ranging from almost 60,000,000 dwelling units to more than 3,000,000 buildings for industry and commerce—altogether 270 different types of structures. Since the end of World War II, some $493 billion has been invested in everything from summer cottages and shopping centers to missile installations.

But all that is prologue.

The expansion ahead, over the next 10 years, will call for structures to serve a population of 220 million people in 1970—a 40 million increase. Our research department believes that the projected growth in the volume of building will bring it to an $85 billion a year level in 1970 as compared with $62 billion in 1959. From now until 1970, analysis indicates that the total anticipated outlay for building can be conservatively put at $600 billion.

This building boom presents a future that has no limit in terms of its potential for the use of prestressed concrete.

As a relatively new material, it is exciting to mention some of the uses to which prestressed concrete is being and will be put.

In addition to basic bread-and-butter fields, the cantilever shell structures that are here, and are to come, open a limitless field.

An even greater use is going to come in buildings where prestressed floors, prestressed panels and prestressed spandrels, will revolutionize building design.

The prestressed concrete industry has, within its control, the power to give to architects the ability to create the "light" look up into the sky. In doing so, it will contribute to the "humanation" of architecture—an architecture that does not deny the importance of the human being by failing to consider that buildings are for people, and, therefore, must be planned, designed and built to embody the visual and aesthetic values of the human scale.

JOURNAL in brief

The Bridge Spanning Lake Maracaibo 12
Riccardo Morandi

With its 771 ft. spans this most spectacular of all prestressed bridges is truly an outstanding engineering (and prestressing) achievement. This imaginative structure is explained in detail with illustrations to serve as a stimulus to designers.

Grouts for Post-Tensioned Prestressed Concrete Members 28
Milos Polivka

This paper gets right into the nuts and bolts of the controversial art of grouting. It is based on the results of 20 years of investigations in this field carried out at the University of California.

Recent British Research on Prestressed Concrete 39
Dr. A. R. Collins

A comprehensive summary of recent British research projects conducted at the Cement & Concrete Association and the Fire Research Organization which are of current interest to all prestressers.

Prestressed Concrete in The Port of New York Authority's Planning 59
John M. Kyle

Based on his experience with a variety of structures the author discusses his problems and solutions in applying prestressing to many diverse areas.

Manhattan's Prestressed Piers 67
Charles Prokop

Engineers and architects will find this article most informative, as this fabricator and field engineer analyzes the sometimes forgotten aspects of structure economics.

Positioning Drape Points in Prestressed Members 74
David Morris

The development of formulae and curves for quickly determining the number and position of hold down points for draped strands is presented.

Prestressed Concrete for Railroads—Past Experience and Future Possibilities 82
F. R. Woolford

Based on thorough studies and continuing research the Railroad Industry has great expectations for prestressed concrete. The author also explains in detail, with illustrations, a unique Russian method of manufacturing prestressed concrete railroad ties, observed during his trip there.

Prestressing Solves Another Building Design Problem 97
K. Yervant Terzian

The application of precast members to a difficult framing plan produced by site and structure requirements is thoroughly discussed.

The Bridge Spanning Lake Maracaibo

by Riccardo Morandi*

Presented at the Prestressed Concrete Institute Convention
New York City September 1960

SYNOPSIS:

This bridge now under construction across Lake Maracaibo is described in the following report, also included is a presentation of some of the principal technical data.

INTRODUCTION:

The problem of building a bridge across Lake Maracaibo does not in itself present grave difficulties, in spite of its excessive length (about six miles), elaborate foundations, and requirement of providing approaches for both the existing and future highway systems. The problem which immediately classifies the work as among the most demanding is the necessity that navigation pass beneath the bridge without limitations to frequency of traffic and, above all, the clearance requirements. Lake Maracaibo, one of the

*Professor, University of Rome
Rome, Italy

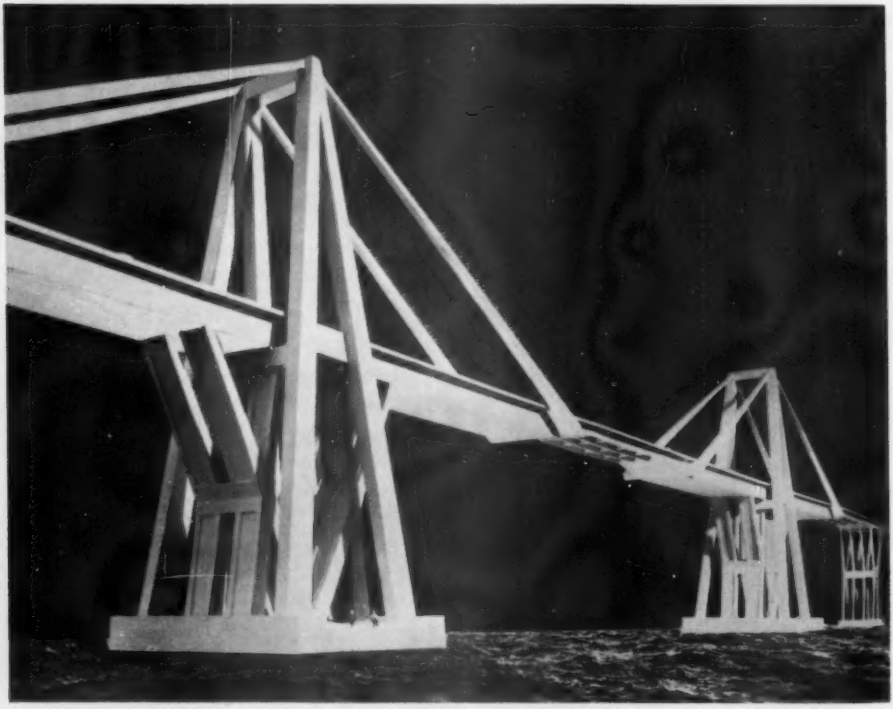

Fig. 1—Model of the Great Span

greatest petroleum centers of the world, has an immense volume of traffic.

The bridge will carry the traffic between the city of Maracaibo (more than 500,000 inhabitants) and the famous Panamerican Highway which runs the entire length of the continent.

The Venezuelan Government, which sponsored the competition for the design and construction of this important work, included in the rider pertaining to the second competion the fundamental characteristics (of the bridge). These characteristics establish the length of approximately six miles with additional requirements that there be, in the navigable channel in the center of the lake a free span with a length of 1,312 feet and a height of 148 feet.

Flanking this great central span there would be five spans, each of a length of 492 feet with the same height of 148 feet. The remaining portions of the bridge could be designed at the engineer's discretion.

The presentation of the theme seemed at first sight to lead to the conclusion that the central part of the bridge must be constructed in steel. The application of this solution. though having its merits, did not present any daring challenges considering the great suspension bridges already executed, especially in the United States.

It was necessary however to consider that the atmosphere of the Lake is particularly injurious to ferrous metals and therefore a bridge in steel would have involved considerable future maintenance expenses.

I asked myself if it were possible to effect this major span using prestressed concrete for its entire length.

The approach spans, 279 feet long, are within the limits of structures of which there are numerous successful examples.

We conceived for the center span a solution in which the span would be completed by means of three elements of already proven dimensions. They would be articulated in such a manner as to fulfill the requirement without resorting to any idea that had not been tried and tested by previous experience.

The Government of Venezuela became convinced through its technical departments that our fundamental solution of the theme was acceptable and it was selected. A most important factor from the economic point of view was that maintenance expenses would be reduced to a negligible minimum.

Subsequently, during the preliminary discussions for the design of the final project, the Venezuelan Government, above all for reasons of economy, modified the fundamental requirements of the project as follows:

—total length: 5 miles and 3,473 feet

—width of the bridge: 57 feet

—the free height above the average level of the lake at the navigable channel: 148 feet

—clear spans for the passage of vessels: 5 each of a length of 656 feet.

DESCRIPTION OF THE PROJECT:

The work was finally designed as follows: Starting from the shore nearest the city of Maracaibo (Punta Piedras):

1 span of 87 feet
2 spans of 153 feet
1 span of 216 feet
15 spans of 279 feet
1 span of 525 feet
5 spans of 771 feet
1 span of 525 feet

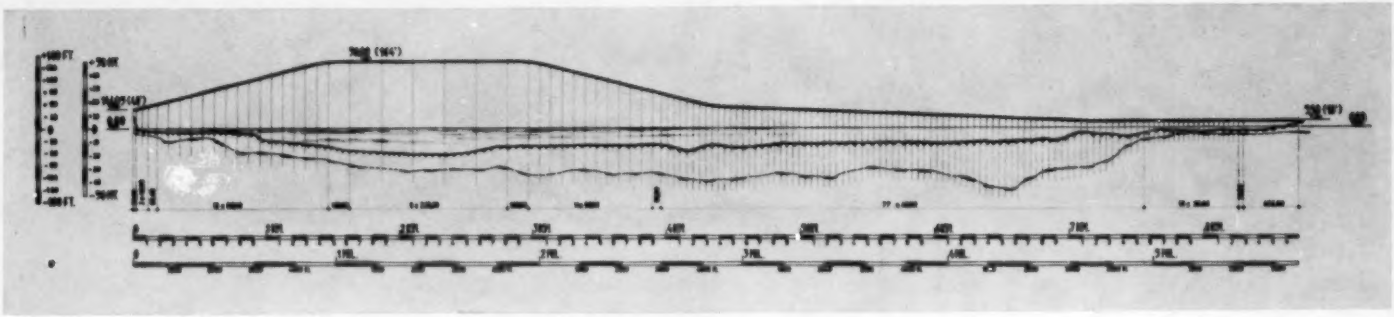

Fig. 2—Profile of the Lake Maracaibo Bridge

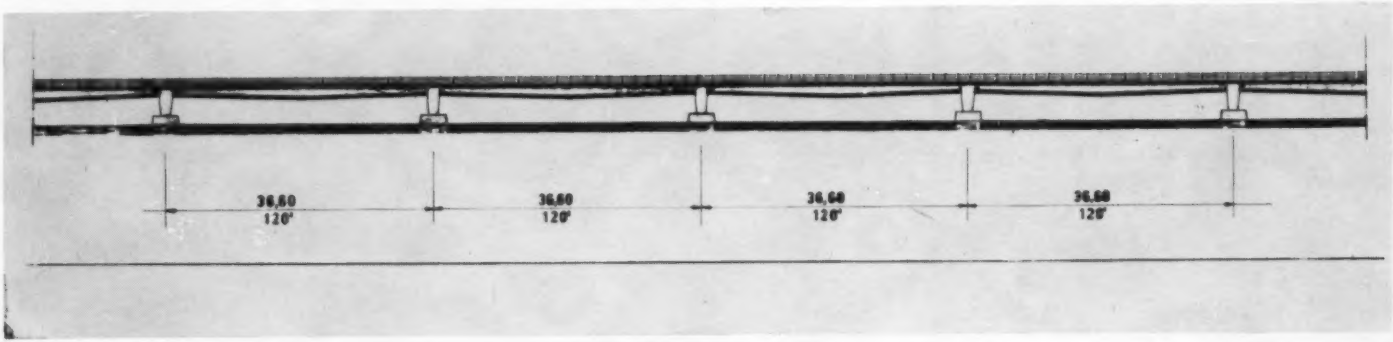

Fig. 3—Elevation 120 ft. Spans

11 spans of 279 feet
1 span of 216 feet
77 spans of 153 feet
20 spans of 120 feet
Approach of 1,335 feet

All of the 153 feet and of 120 feet spans consist of piers with their vertical axes composed of four reinforced concrete walls 2 feet thick. The width varied from 11 ft. 7 in. to 5 ft. 3 in., tied at the top by means of a pillow beam which supports the beam structures. The pier is restrained at its base by a rigid block which in turn is tied to the heads of the foundation pilings which extend to just above the water level of the lake.

Above the piers the horizontal road bed structure is tied at one end by means of a fixed support and at the other by means of an oscillating support. The road bed consists of four beams. They are tied together by the upper slab and by two transverse beams at the supports and by three additional intermediate transverse diaphragms.

The beams of the 151 foot span are characterized by the following dimensions:

—thickness of the slab: varying from 6-¾ in. to 10-⅝ in.

—maximum depth of the ribs: 8 ft. 2-¾ in.

—minimum depth of the ribs: 5 ft. 10-⅞ in.

The quantities of the material are as follows:

—concrete of a minimum cube strength of 6400 lbs. per square inch: quantity 1.23 cu. ft. concrete per square foot of road bed.

—normal reinforcing steel: 4.6 lbs. per square foot road bed.

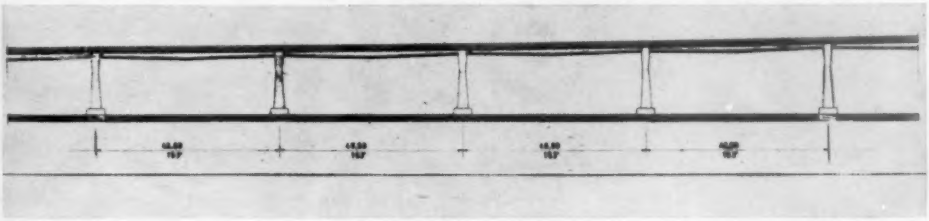

Fig. 4—Elevation 153 ft. Spans

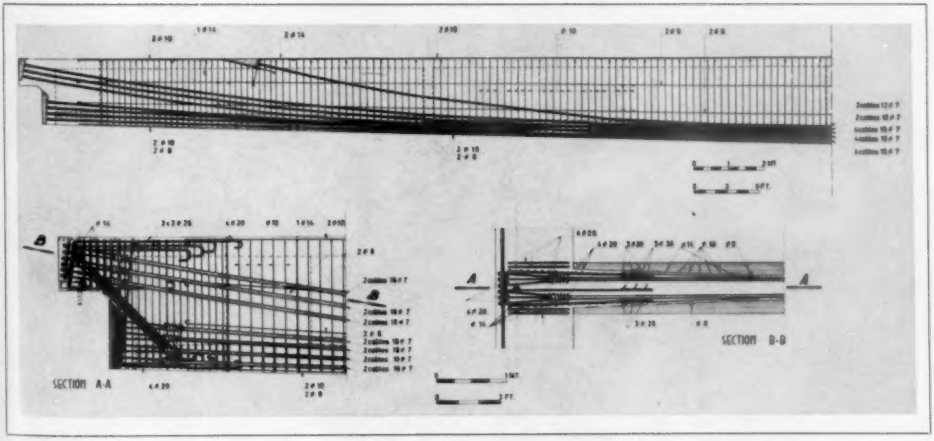

Fig. 5—Details 153 ft. Beam

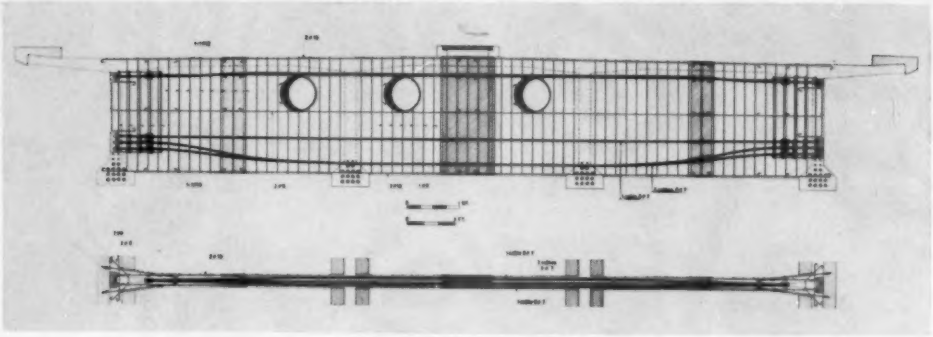

Fig. 6—Transverse Beam

Fig. 7—Construction of 153 ft. Spans

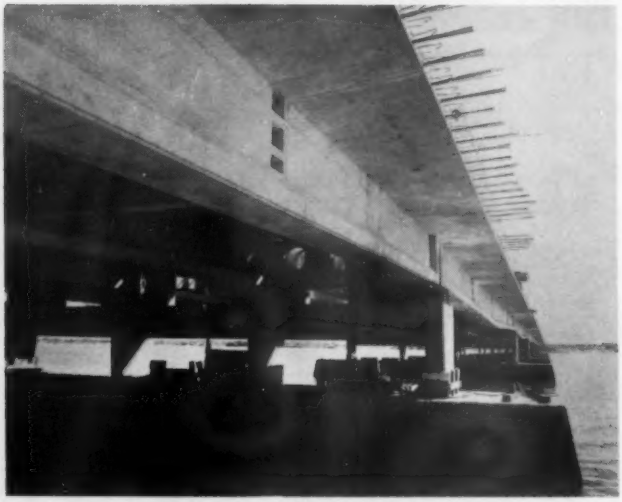

Fig. 8—Construction of 120 ft. Spans

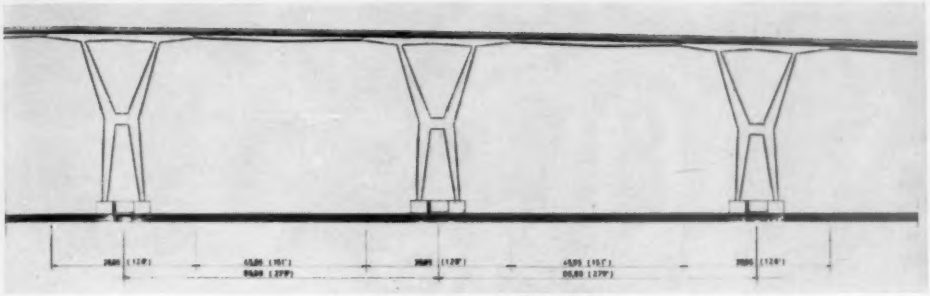

Fig. 9—Elevation 279 ft. Spans

—special prestressing steel, ultimate = 241,791 lbs. per sq. inch, an elastic limit at .2% offset = 206,233 lbs. per sq. inch: quantity 5.35 lbs. per sq. ft. of road bed.

Maximum stresses for the dead weight and live loads are in conformity with the American codes:

f_c = 2134 lbs/sq. in.
f_s = 128,000 lbs/sq. in.

Similarily for the beams of the 118 foot span:

—thickness of the slab: varying from 6-¾ in. to 10-⅝ in.

—maximum depth of the ribs: 6 ft. 6-⅝ in.

—minimum depth of the ribs: 4 ft. 11 in.

The quantities of the material are as follows:

—concrete having a minimum cube strength of 6400 lbs/ sq. in.: 1.18 cubic feet of concrete per square foot of road bed.

—Normal reinforcing steel IIa: 3 lbs. per sq. foot of road bed.

—Prestressing steel ult. = 241,791 lbs/sq. in., an elastic limit at .2% offset = 206,233 lbs. per square inch; 4.70 lbs. per square foot of road bed.

Maximum stresses for dead weight and live loads are in conformity with American Codes:

f_c = 2134 lbs/sq. in.
f_s = 128,000 lbs/sq. in.

The 279 feet spans consist of a special system of trestled piers of the following description:

—A double plate of reinforced concrete placed on top of the foundation pilings and having the following plan dimensions: 52 ft. 4 in. × 14 ft. 11 in. and a height of 9 ft. 10 in., tied together by means of four reinforced concrete beams each 3 ft. 3-¾ in. × 8 ft. 6-⅜ in.

—A special trestle in the form of an "H" with four blades tied at their mid-point and at the top. They have a variable height but are arranged in such a way that the form and the slopes are equal.

—A prestressed concrete beam which ties the two sub-vertical arms of each trestle. This 128 ft. 1-½ in. long beam extends as a cantilever and supports at each end a 151 ft. drop in beam. (See Fig. 9)

The complete trestle consists of an elastic system calculated for:

—dead weight of the structure
—live loads
—wind
—temperature variation
—conditions of construction and assembly.

The maximum stresses are found to be as follows:

—for the parts in normal reinforced concrete:

f_c = 1422 lbs/sq. in.
f_s = 22,757 lbs/sq. in.

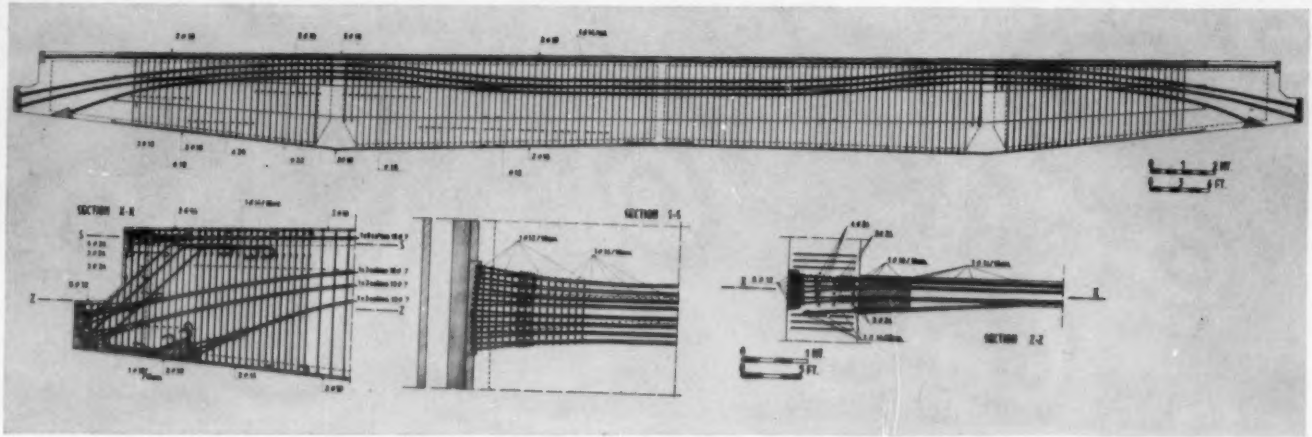

Fig. 10—Cantilever Beam for 279 ft. Span

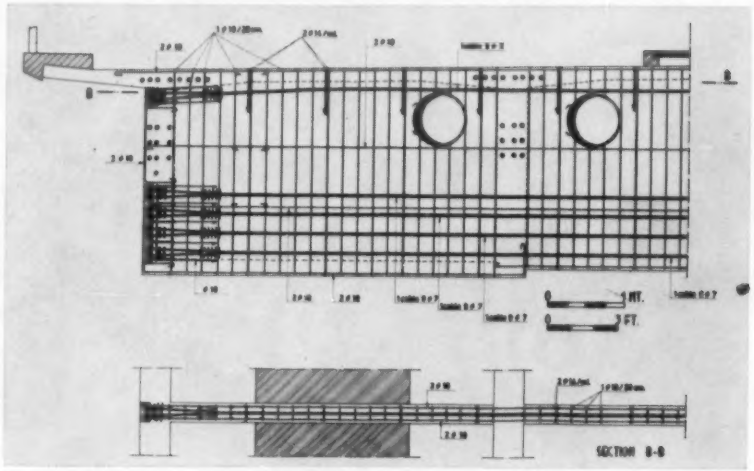

Fig. 11—Transverse Beam

–for the parts in prestressed concrete:

$f_c = 2133$ lbs/sq. in.

$f_s = 128{,}000$ lbs/sq. in.

The quantities of material employed for the road bed (excluding the trestles) are:

–Concrete: 1.80 cubic foot per square foot of road bed.

–Normal reinforcing steel: 8.78 lbs. per square foot of road bed.

–Special prestressing steel: 5.66 lbs. per square foot of road bed.

The 771 foot Spans

This span consists of a complex of six beams, each 620 ft. 3 in. long. They were designed as continuous beams with six restraints of which four are governed by the oblique struts intersecting each other in the form of "XX". The other two restraints are the two ends supported by the tension member that ties them and passes above the double antenna system in the form of an "A". This "A" frame is supported by a great plate placed at the level of the lake, which is also the base of the entire pier complex which in its turn is supported by the foundation pilings. (See figure 13)

The free ends of each continuous beam are tied by simply supported drop-in beams having a theoretical span of 151 feet, identical to those used for all the other spans.

The base plate, the "A" antenna system and the "XX" frames are all constructed in normal reinforced concrete.

The continuous beam is prestressed internally by steel cables and externally by the horizontal component of the oblique antenna tension members. The latter are prestressed to compensate for the tension caused by the dead load deflection of the cantilever beams. This means that the only lowering of this support is that caused by moving live loads. This was accomplished by determining the lengthening of the tension member due to the dead weight and then prestressing to compensate for it.

For the investigation of the tension members we examined the influence of the elastic deformation of the antenna in the case of asymmetric live loads which also determine asymmetric reactions on the antenna itself. A similar investigation was carried out for the temperature variations of the system consisting of the antenna tension members and beams.

To give an idea of the range of stresses in the cables, their maximum and minimum values are as follows:

–minimum tensile stress: 6027 tons

–maximum tensile stress: 6586 tons

The tension members consist of patented cables developed by the firm of Felton and Guilleaume.

A member of particular interest is the transverse beams placed at the ends of the cables whose function is that of distributing the reaction of those cables uniformly across the entire width of the road bed. In these we observe extremely high shear forces for which we compensated by placing a great number of prestressed cables with sharp curvature.

The beam system, length of 620-ft. 3 in. (see fig. 14) is a box structure with high transverse rigidity. The thickness of the box's components are, an upper slab varying from 6-¾ to 10-⅝ in. walls and the lower slab of a minimum thickness of 9-⅞". This is supported by a "X" sub-structure with its restraints con-

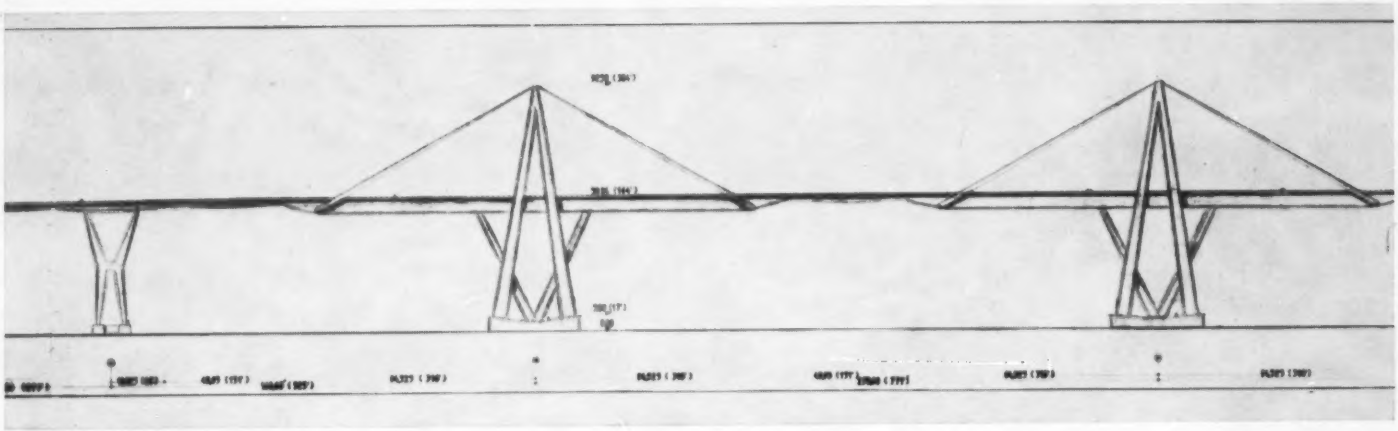

Fig. 12—Elevation 771 ft. Spans

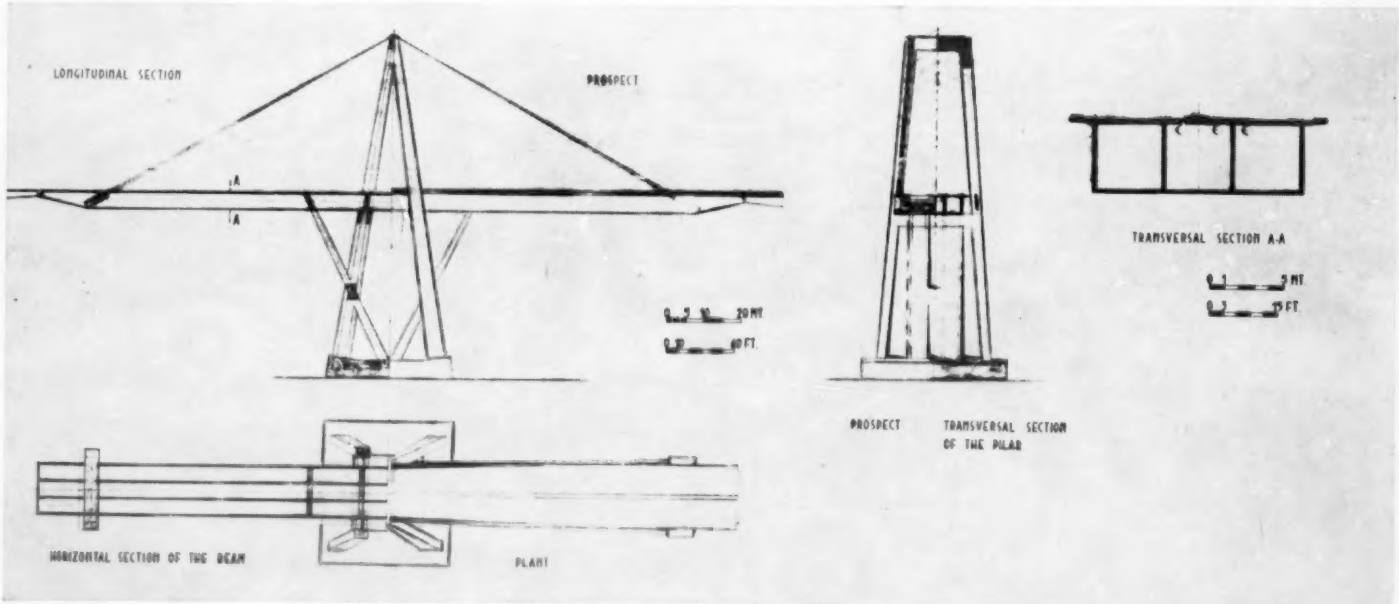

Fig. 13—Details of 771 ft. Span

sidered as connections with respect to the distribution of the bending moments along the length of the beam itself.

The maximum stresses found in the beams are identical to those already described for the other minor beams—as follows:

—compression:
$f_c = -2134$ lbs/sq. in.
—tension:
$f_c = +114$ lbs/sq. in.
$f_{steel} = 128{,}000$ lbs/sq. in.

The quantities of material are as follows:

—Concrete of a minimum cube strength of 6400 lbs/sq. ft.: 3.28 cubic feet per square foot of road bed.
—Normal reinforcing steel: 12 lbs. per square foot of road bed.
—Prestressing steel, ult. = 241,781 lbs/sq. in., elastic limit at .2% offset = 206,233 lbs. per square inch: and a quantity of 4.90 lbs. per square foot of road bed.

The "XX" system, constructed of reinforced concrete with characteristics similar to those in the "H" system, was calculated as an elastic system with following hypotheses:

1st phase: stresses in the system with hinged joints between beam and pier.
2nd phase: stresses induced by the actual rigid tie between the beam and pier.

Conditions of Loading:

—dead weight of the structure
—live loads
—shrinkage
—braking
—temperature variations
—wind

The following stresses resulted:
$f_c = 1707$ lbs/sq. in.
$f_s = 25{,}602$ lbs/sq. in.

THE FOUNDATIONS:

Lake Maracaibo has a depth of water variable from a maximum of 52 ft.-6 in. (for the navigable channel) to a minimum of 6 ft.-¾ in. along the projected crossing.

The lake floor consists of slimey sand. At a depth variable from a maximum of 121 ft.-5 in. below the average level of the lake to a minimum of 32 ft.-10 in., the solidity of the sandy terrain increases. It was agreed to define as firm ground, soil that which resists 20 blows of a standard penetration test.

It was determined that at 32 ft.-10 in. below firm ground the resistance of the standard penetration test increases to 50 blows. It was finally agreed to consider the piles fixed to two meters below firm ground. The notable variation in the depth of the lake and in the depth of the firm ground directed us towards diverse solutions for the various foundations of the bridge.

At the zones nearest the shores and therefore of lesser depth it was sufficient to resort to short piling, using normal prefabricated piles of a maximum safe bearing capacity of 300 tons.

For the deeper foundations on the other hand, larger piles were studied with larger bearing capacities (variable from 500 to 700 tons) built as follows:

A metal sleeve of a diameter of about 4-½ feet is driven into the floor of the lake to the required depth, in some cases 150 feet below the level of the lake. Then a prefabricated pile of a slightly smaller diameter than that of the metal sleeve is introduced into the sleeve. This pile consists of a hollow cylinder with 7 inch thick walls. Finally, the metal sleeve is extracted and cement is injected into the zone between

the ground and the pile, completely saturating this area.

Every foundation pile group is tied together by a large plate of reinforced concrete. This footing is designed to transmit the horizontal and vertical actions derived from the superstructures in a uniform manner upon all the piles of the foundation. The foundation for the major spans are still in experimental stage to determine their resistance to extremely high horizontal forces.

THE SYSTEM OF PRE-STRESSING:

All prestressed structures of the road bed, the beams and slabs, have been prestressed using the Morandi system. (see fig. 14) I have used this system, commonly known in Europe as the Italian system, in numerous important structural works and I have had a successful experience with it for more than 12 years. I review briefly here the characteristics of the system which was designed to satisfy the following requirements:

1) To impart equal tension to each of the single wires of the cable.
2) The possibility of realizing, with the same system of anchorage, cables of diverse sections.
3) The possibility of applying the prestressing by degrees, and to recalibrate the tensions and therefore greatly facilitate the unlocking.
4) The possibility of determining the exact tensions in the wires without unlocking them, for the following purposes:
   a) The immediate verification of the tension imparted in order to control eventual losses of tension during locking.
   b) Deductions from tensions read owning to friction which can be verified along the cable.
   c) Verification of the tensions before the recalibration in order to unlock only those wires which require correction, giving in general a control of the performance of the plastic flow of the material.
5) Security of locking and high resistance to oscillating forces.
6) Lightness and economy of the anchorage.

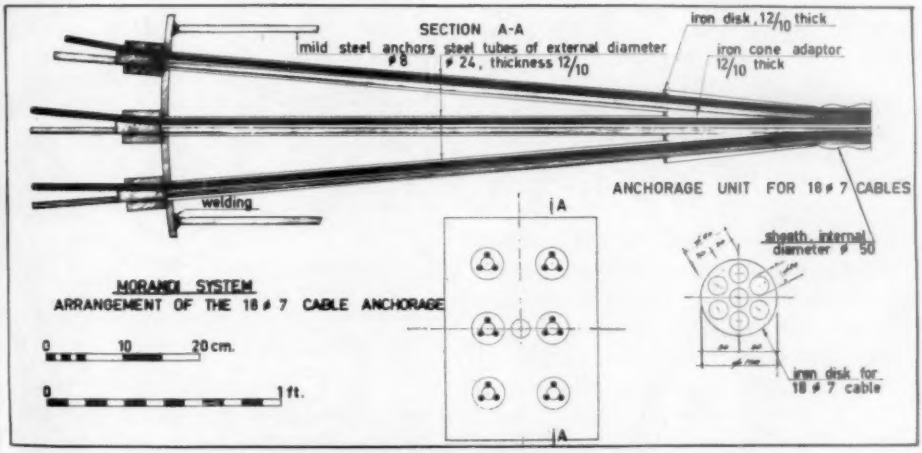
SECTION A-A
mild steel anchors ø 8
steel tubes of external diameter ø 24, thickness 12/10
iron disk, 12/10 thick
iron cone adaptor 12/10 thick
ANCHORAGE UNIT FOR 18 ø 7 CABLES
welding
A
sheath, internal diameter ø 50
MORANDI SYSTEM
ARRANGEMENT OF THE 18 ø 7 CABLE ANCHORAGE
0 10 20 cm.
0 ft.
iron disk for 18 ø 7 cable
A

Fig. 14—Anchor Details of Prestressing System

7) Rapidity of the tensioning operations and ease of handling of the equipment.

The tensioning and locking equipment consist of:

a) A light horizontal jack having an anular compression chamber for the tensioning of three wires at a time. The weight of a complete jack for tensioning three cables of 5 mm diameter is 26.5 lbs. for tensioning 3 cables of 7 mm diameter, 48.5 lbs.

b) A lock for 3-5 mm or 3-7 mm cables, consisting of a hard steel cylinder in whose bore are introduced the three steel wires. Along the walls of each bore are grooved three channels, in each of which one wire of the trio is accommodated. In the bore of of the cylinder between the 3 wires, a pin of soft steel, having along its sides 3 grooves of a convergence exactly equal to the canals of the cylinder and in a coincident position.

c) Anchorage plates to distribute the forces transmitted to the cylinders at the head of the structure, with as many 20 mm diameter holes as there are three-wire groups comprising the cable. Plus a subsidiary hole to permit the injection of grout along the cable.

The following therefore is the sequence of the prestressing operations. After having fixed the terminal plates into the concrete at the ends of the structure, the anchorage block for each group of three wires is introduced, the pin is fixed into the block and the jack is operated after the three wires have been introduced into the anular chamber. Thereupon the three wires are firmly grasped at the head of the jack by means of a specially designed pin. In view of the extreme ease of operation of the equipment it is possible to proceed in a very few seconds to the stressing and locking of the three wires.

The jack is provided with a special movable ring nut having the function of holding the cylinder against the plate (during the stressing operation), or, to free the cylinder in order to shift it back along with the wires (operation of direct reading of the tensions), or, finally, to relock the cylinder to the plate for the eventual successive operation of unlocking of the wires and recalibration of the tensions (operation of checking of tensions).

CRITERIA FOR CALCULATION OF THE PRE-STRESSED STRUCTURES

I)—Calculation of friction losses:

Starting from the point of anchorage the percentage of the decrease of the tension is calculated taking into account the normal friction forces as well as the effects of friction in the curved portions and the losses of tension in the anchorage.

The coefficients of friction were assumed in accordance with the Italian Code and are as follows:

nature of the sleeves or surfaces surrounding the cables	coefficient of friction in a straight line fd 1000	coefficient of friction in curve fc
smooth concrete:	5	0.5
sheet metal:	2.5	0.3
lubricated sheet metal:	1	0.2

The results obtained are:

A loss of tension owing to friction up to mid-span for the 151′ beam (metal sleeve): about 16%. Of the total loss about 6% is concentrated

at the anchorage.

II)–Calculation of tension because of plastic flow and shrinkage of the concrete:

Given the section of concrete, let:

ϵ_s = unit deformation for shrinkage.

μ_E = final value of the plastic deformation of the concrete.

E_c = modulus of elasticity of the concrete.

e_v = distance of the center of gravity of the cables from the center of gravity of the ideal section (concrete + steel).

J_v = distance of the center of gravity of the concrete section from the center of gravity of the ideal section.

e = distance of the center of gravity of the cables from the geometric center of gravity of the concrete section.

Given:

$$\alpha = n\,A_f \left(\frac{1}{Ai} + \frac{e_v^2}{Ji} \right)$$

(A_f = total area of the cables in the section under consideration). With the hypothesis that the curve $\epsilon_s = f_1(t)$ is analogous to that $\mu_t = f_2(t)$ it is possible to evaluate the loss of tension due to shrinkage along with that due to the plastic flow of the concrete. Finally we obtain the variation ΔX of the force of prestressing:

$$\Delta X = nA_f \left[\frac{X' \frac{\alpha}{n A_f} - \frac{Mg}{J_i} e_v - \frac{Ng}{A_i} + E_c \frac{\epsilon_s}{\mu_\epsilon}(1-\alpha)}{\frac{1}{\mu} + \frac{\alpha}{2}} \right]$$

The plastic deformation of the steel which produces the total tension losses of 7% takes place during the same period in which the slow deformation of the concrete occurs.

For this reason the average loss will be valued in the measure of 3.5% in the above formula for which: $X'_o = 0.965\ X$ in which X is the force of pre-tensioning at the time $t = 0$.

Mg and Ng are stresses due solely to the permanent loads (exclusive of pre-stressing and live loads).

Isostatic systems: The relation is exact for every section.

Hyperstatic systems: We have a good approximation in the case in which the static system is not modified after prestressing.

In the hyperstatic system we must also consider M and N as coming from the hyperstatic effects of prestressing.

For the 151′ long beam, the total loss of tension due to creep in the steel, and plastic flow and shrinkage of the concrete is 19%.

III)–Determination of the effective forces of prestressing in any section at the time $t = 0$ and $t = \infty$.

Based upon the maximum stress admissible at the anchorage, we proceed to the verification of the stresses at the sections, taking into account that the most unfavorable conditions of stressing are:

$t = 0$: Minimum moment relative to the time of construction when, the force of prestressing X′=X max.

$t = \infty$: Maximum moment that can be realized during the structures lifetime with the force of prestressing X′=X min.

METHODS OF CONSTRUCTION:

Concerning the construction methods, summarized below are the general concepts:

For the sector of the bridge lying towards Punta Jguana, including all the lesser spans whose supports vary in height above the level of the lake, we proceeded by launching (by

Fig. 15—Pier Construction 279 ft. Spans

Fig. 16—Pier and Beam Construction 279 ft. Span

means of floats) two beams which constituted half of the width of the roadbed structure.

The four beams, separately prefabricated, were brought together two at a time on the vessels which transported them into their position with respect to the piers. The floats were sunk which transferred the support of the beams to the piers.

In this way the only poured in place operation was that of the central portion of the slab after the placing of the transverse prestressing cables in the slab.

A similar method of execution was followed for the 151 feet beams, with the only variation that it was necessary to resort to a system of cranes in order to raise the beams from the level of the floats to the level of the supports.

On the other hand, as far as all the piers and trestles in the form of an "H" and double "X", these are being poured in place with the use of metal forms and centerings in tubular steel. For the pouring of the large beams of the major spans, specially trussed steel centerings will be hung by means of temporary tension rods from the antennae from which the final tension members will hang.

At the present moment, we have already built about 30 spans towards Punta Jguana and 5 spans towards Punta Piedras. The rest of the foundations are presently in construction while all the prefabricated beams are in the last phase of construction.

It is estimated that the bridge will be completed in the first part of 1962.

GROUTS FOR POST-TENSIONED PRESTRESSED CONCRETE MEMBERS

by Milos Polivka*

Presented at the FIP-RILEM Symposium on
Injection Grout for Prestressed Concrete
Trondheim, Norway, January, 1961

INTRODUCTION

The desirability of grouting the tendons of cables in post-tensioned prestressed concrete members is well recognized. The two main objectives of grouting are to protect the steel from deterioration due to corrosion and to provide bond between the tendons and the concrete member after tensioning. Both of these requirements call not only for a complete filling of the void space within the post-tensioning units but also for a properly designed grout mixture possessing desired fresh and hardened properties. Grouts employed must be of sufficiently low viscosity to permit easy injection; should be of low water content to insure high strength and low shrinkage characteristics; and should be of very low bleeding characteristic to prevent segregation and formation of water pockets under the tendons or in the top of the cable. They should also contain a gas-forming agent which will cause the fresh grout to expand (if unrestrained) during the first few hours and develop pressures when restrained against expansion. When inlets and outlets are closed after grouting, the pressures developed reduce the size of entrapped air bubbles in accordance with Boyle's Law. Grouting equipment and the grouting procedures employed are also of great importance in insuring a complete filling of the void spaces in the cables.

Too little attention is being paid in practice to proper grouting procedures of post-tensioning units. There have been reported some failures of prestressed concrete members resulting from corrosion of tendons due to the use of improper grouts or grouting techniques.[1] Malpractice of the art of grouting can frequently be observed in the field. Current American specifications and recommended practices for grouting post-tensioned prestressed concrete [2,3,4] are of little practical value. They contain only very brief statements on the various phases of grouting and are of no real value to the uninformed engineer or contractor.

This paper, which discusses the various factors concerning grouts for post-tensioned prestressed concrete members, is based on results of many investigations in the field of cement grouting carried out at the University of California over the last 20 years.[5,6,7]

COMPOSITIONS OF GROUTS:

Cement grouts are composed of cementing material, water, chemical admixture, and sometimes fine sand. Neat cement grouts are those containing cementing material and water. Sand-cement grouts are those containing in addition to the cementing material and water a fine sand. Both of these types of grouts will usually also contain a chemical admixture.

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Whether a neat cement grout or a sand-cement grout should be used will depend on the type and size of cable employed and on the size of the tendons. The use of sand-cement grouts is particularly desirable because of their lower cement content and therefore lower shrinkage characteristics, which is in the interest of sustained high bond. In the Magnel cable for example, the void spaces between wires are large enough to permit the use of a sand-cement grout; whereas the void spaces in some of the Freyssinet or Strescon cables are so small that a neat cement grout has to be used.

It is also desirable to employ in the grouts a chemical admixture or admixtures of proven reliability which will improve plasticity and decrease the tendency towards bleeding of fresh grout, will retard its rate of stiffening, will decrease mixing-water requirement, and will cause the grout to expand (if unrestrained) for a period of several hours after mixing.

Inasmuch as the composition of individual grout materials and their proportions significantly affect the properties of fresh and hardened grouts, laboratory tests should be conducted prior to their use in the field.

GROUT MATERIALS

CEMENTING MATERIAL

The cementing material used in grouts for grouting of post-tensioned concrete members may be an ordinary portland cement, a high-early strength cement, a portland blast-furnace cement, or a portland pozzolan cement.

To increase the water retentivity (reduce bleeding) of the grout and thus reduce the tendency towards separation of its solid constituents, particularly in sand grouts, the use of a material which is more finely divided than the ordinary portland cement is desirable. The objective is to produce a fresh grout for which the solids will remain in suspension with practically no bleeding. This will eliminate the possibility of formation of water voids, which may later become air voids in contact with the steel.

Such materials, used as additions to portland cement include the pozzolans, granulated blast-furnace slag and natural cement. A pozzolan is a finely divided siliceous or siliceous and aluminous material which, when used as an addition in a cement grout chemically reacts with calcium hydroxide (a product of portland-cement hydration) to produce cementitious compounds. The pozzolans in themselves possess no cementitious value. Natural cements and granulated blast-furnace slags are cementitious in their own right.

The amounts of any such finely divided material to be added to the portland cement will depend on the nature of the material itself and on the mix proportions of the grout.

In the following tabulations are listed some of the finely divided active mineral materials along with the range of amounts in which they usually would be used as an additions to the portland cement in grouts.

Type of Material	Addition to Portland Cement, % by weight
Diatomaceous Earth	3 - 6
Calcined Opaline Shales	10 - 20
Natural Cement	25 - 35
Fly Ashes (of high fineness and low carbon content	25 - 35
Water-Quenched Blast-Furnace Slag	30 - 50

Other pozzolans are discussed in a paper by Raymond E. Davis.[8]

Such materials should be used only if they improve such properties of fresh grout as low bleeding, easy pumpability and penetrability of small voids, and also if the hardened grout is of adequate strength. Their most important contribution to the properties of grouts is their effect on reduction of bleeding and segregation. In neat cement grouts these additions would be used in a smaller percentage than in the sand-cement grouts. Their fineness will also influence the optimum percentage in which they should be added to portland cement. The finer they are the smaller will be the amount needed to achieve the desired effects.

Sand—Except as to grading, the sand used in grouts should meet the same requirements as those sands employed in concrete. Where several sands are available, preference should be given to the one which tests indicate will produce optimum results with regard to such properties as water requirements, bleeding, and strength of hardened grout. All of the sand should pass the No. 30 sieve (0.59 mm). A suggested range of grading limits is shown in the following tabulation.

Sieve		Percent
U. S. Std. No.	Opening, mm	Passing
30	0.590	100
50	0.297	40 - 60
100	0.149	15 - 25
200	0.074	0 - 5

The sand grading to be used will depend on the size of the void space to be grouted. Coarser grading should be used for large voids and finer grading for smaller void spaces.

CHEMICAL ADMIXTURES

The two principal types of chemical admixtures widely utilized in grouts are the gas-producing agents and the water-reducing retarders. There is no advantage to be gained through the use of any of the air-entraining agents commonly employed in conventional concrete construction. Examples of the use of chemical admixtures, their advantages and disadvantages, and considerations involved of their use in mortars and concretes are cited in the report of ACI Committee 212 on Admixture for Concrete.[9]

Gas-Producing Agents—Aluminum is one of the metals which is capable of rapidly reacting in alkaline solutions, such as occur in mixtures containing portland cement, to produce hydrogen gas. There are also some chemical compounds capable of gas generation. For the grouts employed in prestress concrete work, the use of aluminum powder is recommended.

The aluminum powder should be used in quantity sufficient to produce an unrestrained expansion of about 10 percent of the volume of the grout prior to the time of setting. At normal temperatures and with cements of normal alkali content, for an aluminum powder of the proper grade most of the hydrogen gas generation which produces the expansion will take place within the first three to four hours after mixing. The amount of aluminum powder required to produce a 10 percent expansion of grout will range from 0.01 to 0.03 percent of the weight of cementing material. For proper dispersion of such a small quantity of aluminum powder in the grout mixture, the powder should first be blended with cement or other finely divided material in proportions of say one part of aluminum powder to one hundred parts of cement. The material can then be packaged or

batched in appropriate amount for quantities such as one sack of cement.

Since there are many different commercial grades of aluminum powder available, their suitability as a gas-forming agent should be determined by testing them in grouts containing the proposed job materials. In Fig. 1 are shown the results of such an evaluation of six aluminum powders in a grout mixture containing five parts of cement, two parts of fly ash, and six parts of sand by weight at a water-cementing material ratio W/(C + F) of 0.44 by weight. Aluminum powder samples 1 to 3 were leafing types and samples 4 to 6 non-leafing types. The fineness of these aluminum powders varied in order of their numbers from 31,000 to 11,000 sq. cm. per gram, No. 1 being the finest and No. 6 the coarsest. All powders were employed in the amount of 0.015 percent by weight of cementing material. The grouts also contained a water-reducing retarder in the amount of 0.20 percent by weight of cementing material. The desired 10 percent unrestrained volumetric expansion of the grout was reached within 2 to 4 hours after end of mixing with the three leafing type aluminum powders (samples 1, 2, and 3).

The alkali content and perhaps the compound composition of the cement also has a large effect on the expansion characteristics of grouts.

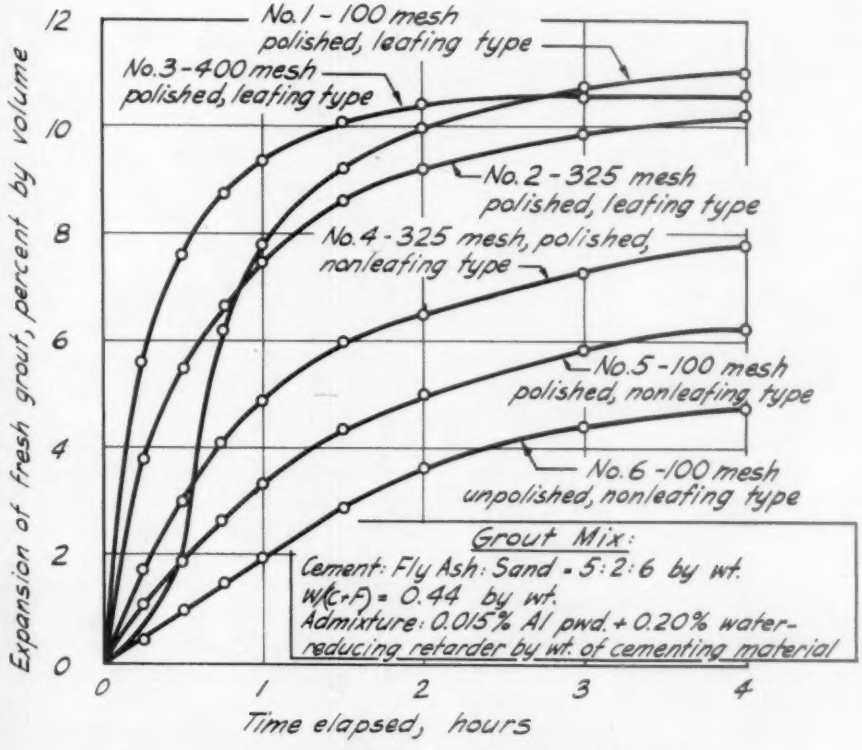
No. 1 - 100 mesh polished, leafing type
No. 3 - 400 mesh polished, leafing type
No. 2 - 325 mesh polished, leafing type
No. 4 - 325 mesh, polished, nonleafing type
No. 5 - 100 mesh polished, nonleafing type
No. 6 - 100 mesh unpolished, nonleafing type
Grout Mix:
Cement: Fly Ash: Sand = 5:2:6 by wt.
W/(C+F) = 0.44 by wt.
Admixture: 0.015% Al pwd. + 0.20% water-reducing retarder by wt. of cementing material
Expansion of fresh grout, percent by volume
Time elapsed, hours
0
2
4
6
8
10
12
0
1
2
3
4

Fig. 1—Effect of type of aluminum powder on expansion characteristics of fresh grouts at 70°F

Tests conducted on grouts containing cements of different alkali contents showed as much as 50 percent variation in total expansion. It was also observed that the rate of reaction is less rapid for the unpolished aluminum powder and also the magnitude of grout expansion is less.

Water-Reducing Retarders—The use of a water-reducing retarder which will reduce the water content and retard the rate of stiffening of a grout mixture is highly desirable. Water-reducing retarders which are suitable for this purpose include calcium, sodium, and ammonium salts of some of the ligno-sulphonates, some of the organic acids including those of the hydroxy group and the amine salts of these acids. For use in prestress work any water-reducing retarder should be free from calcium chloride or compounds which might contribute to corrosion of the steel.

Some of the commercially available grouting admixtures contain both a gas-producing agent and a water-reducing retarder. Some even contain a thickening agent which tends to stabilize the grout suspension, reducing the tendency towards bleeding and segregation. Because of the complexities involved in developing such multi-purpose chemical admixtures, it will usually be cheaper and better to employ a commercially available admixture specially designed for the purpose of grouting post-tensioned concrete members.

MIX PROPORTIONS

The proportions of grouting materials should be based upon results of laboratory tests made on fresh and hardened grouts prior to their use in the field. The amount of mixing water employed should be such as to produce a grout having the consistency of a thick cream or heavy paint. When permitted to stand until setting takes place, the grout should exhibit practically no bleeding or segregation and should expand not less than 6 nor more than 12 percent of its original volume. For neat cement grouts containing a water-reducing retarder, the water-cementing material ratio will be in the neighborhood of 0.40 to 0.45 by weight. When a sand-cement grout is employed for which the absolute volume of sand is approximately equal to the absolute volume of the other solid constituents of the grout, the water-cementing material ratio by weight will be in the order of 0.45 to 0.55.

Evaluation of mix proportions and of grout materials should be made on grout mixtures of fixed consistencies. Grout consistencies, in terms of a flow factor, can be measured by means of the "flow-cone method," later described under "Testing Methods". Test data indicate that a grout consistency suitable for grouting of prestress cables is represented by a flow factor of about 20 to 24 seconds for neat cement grouts, and about 24 to 28 seconds for sand-cement grouts.

Adjustment of mix proportions might also be necessary to produce grouts of desired expansion characteristics, absence of bleeding and satisfactory strength. The variables can include the type of cement, richness of mix, the type and amount of finely divided active mineral admixture (pozzolan, natural cement, or granulated blast-furnace slag), the quantity and grading of sand, and the kind and amount of chemical admixture.

TESTING METHODS FOR GROUTS

The important properties of fresh grouts to be evaluated are consist-

ency or viscosity and the expansion and bleeding characteristics. The compressive strength of hardened grout should also be determined.

For laboratory tests of grout properties, it is advisable to adopt a standardized mixing procedure. Such factors as type and speed of grout mixer, manner of charging the mixer, and time of mixing will have an influence on the resulting grout properties.

Laboratory Mixing of Grout

The mixer shown in Fig. 2 has been found to be very satisfactory for laboratory tests. The following mixing procedure has long been employed in our laboratory at the University of California.

The estimated quantity of water necessary to produce a grout of desired consistency is placed in the mixing vessel, the motor is started, and the reostat is adjusted so that the speed of stirring is about 500 rpm. The cementing material is then introduced gradually into the mixer over a period of 1 minute. The time of starting this operation is considered as the start of mixing. As the cementing material is fed into the mixing vessel, the speed of stirring is increased gradually until the maximum speed (about 8,000 rpm) is attained. Three minutes of mixing is sufficient to produce a homogenous mixture. If the grout is to contain sand, the sand is introduced after 3 minutes of mixing, gradually pouring it into the mixing vessel over a period of about ½ minute, and continuing the mixing for an additional 2½ minutes. The total elapsed time in this case is 6 minutes from the time of starting.

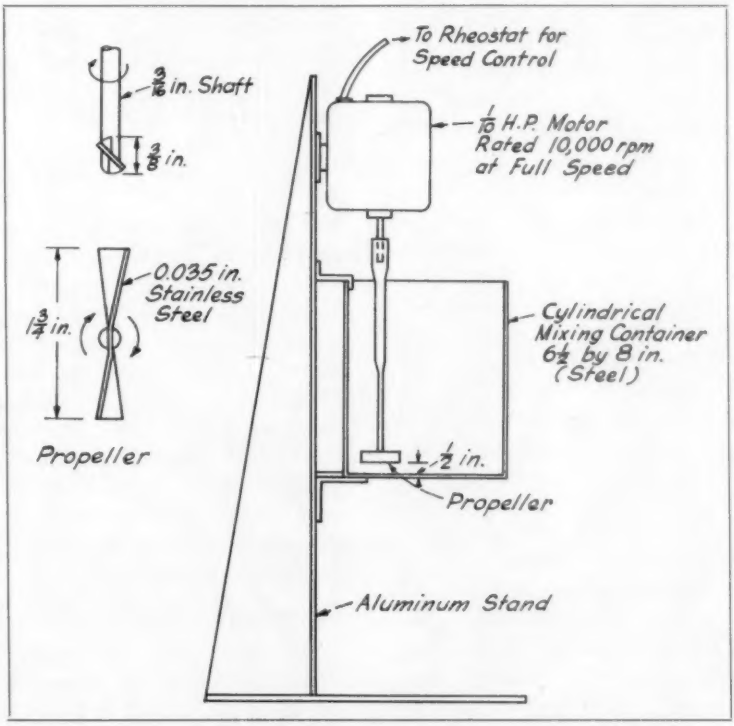

Fig. 2—Laboratory grout mixer

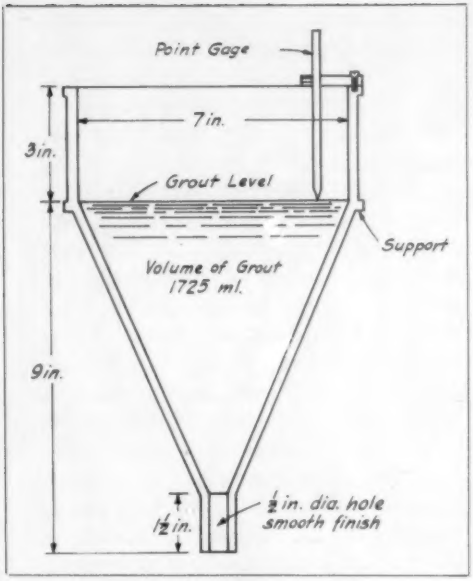

Fig. 3—Flow Cone apparatus

Consistency Determination

The consistency, or essentially the pumpability, of the grout is measured by the resistance of flow through a pipe or an orifice. A convenient method for evaluating consistency is by the measurement of the time of efflux of a fixed quantity of grout, under falling head through a standardized discharge tube. For this purpose a flow-cone apparatus having the shape and dimensions shown in Fig 3 is utilized. The consistency is determined as the time required for a given volume (1725 ml) of grout to flow from the cone through the discharge tube. This time of efflux of the grout, in seconds, is called the "flow factor". The flow cone should be moistened prior to use by flushing it out with water and allowing it to drain for about 1 minute, before the grout for test is to be poured into the flow cone. During filling of the cone with grout, the discharge tube is sealed by placing a finger over the end. Immediately after filling of the cone, the grout is allowed to discharge by quickly removing the finger; and a stop watch is started simultaneously. The watch is stopped at the first break in continuous flow of grout from the tube.

Measurement of Expansion

The expansion of a sample of grout is determined as the change in volume of a grout column between the original level and the level at the end of a specified period. For this test approximately 900 ml of freshly mixed grout is poured into a standard 1000-ml graduated cylinder, and the volume is observed and recorded. The expansion of the sample is periodically observed. Depending upon the type of grout tested, the test may be discontinued at either 3 or 4 hours when its evident that expansion has practically ceased. At the end of this period, the bleeding water, if any, is poured from the surface of the grout into a small graduated cylinder, where its volume is observed.

The grout expansion is reported as percent expansion based on original grout volume. As discussed earlier, it is generally considered that a total expansion of 10 ± 2 percent is desirable. If there is any bleeding water on the surface of the grout, it should be collected, measured, and reported as percent of bleeding based on original volume of grout. For the type of grouts considered here, it is desirable that they exhibit practically no bleeding. In no case should the use of a grout exhibiting bleeding in excess of 0.4 percent be permitted.

Compressive Strength

In evaluating the compressive strength of a grout, it is difficult to simulate the conditions to which the grout is subjected in the post-tension-

ing cables. It was found that a suitable method for manufacture of grout specimens is to use heavy steel molds provided with end plates and rods that will insure complete restraint of the grout after specimens are cast. At the University of California 3 by 6-in. molds, with their bottom plate in position, are filled with grout and immediately sealed by fastening the top plate securely with the rods. To insure a complete seal, the end plates should be provided with suitable gaskets. The tendency for the grout to expand will develop a pressure similar to that expected to be developed in the post-tensioning cable. In Fig. 4 are shown the pressures developed by a grout mixture containing five parts of cement, two parts of fly ash, and six parts of sand by weight at a water-cementing material ratio of 0.44 by weight. The grouting admixture contained 0.015 percent of a water reducer by weight of cementing material. Three hours after casting, the grout developed a pressure of 28 psi.

Restraint of the grout has a significant effect on the compressive strength. To evalute this effect, a neat cement grout containing ten parts of cement and one part of fly ash at a water-cementing material ratio of 0.40 by weight was used for manufacture of restrained and unrestrained grout specimens tested in compression at various ages. This grout contained 0.015 percent aluminum powder and 0.20 percent of a water-reducer by weight of cementing material. Its free expansion was 11.0 percent by volume. The test results obtained are shown in Fig. 5. The specimens for the restrained condition were kept in sealed 3 by 6-in. steel molds up to the age of test. The unrestrained specimens were cast in 3 by 6-in. cardboard molds with an open top. The grout which rose above the top of the mold was cut off four hours after casting of the specimens. The 28-day compressive strength of the unrestrained specimens (1870 psi) was only about

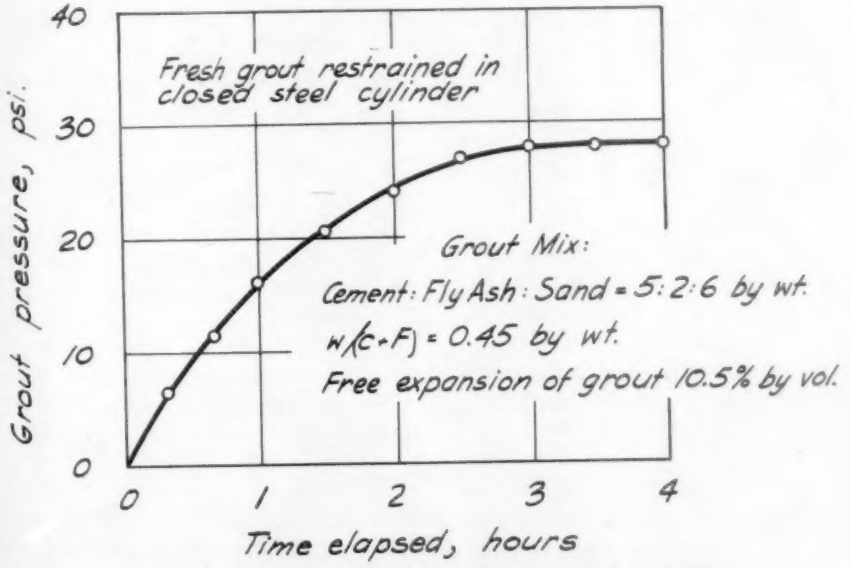

Fig. 4—Pressures developed due to volumetric restraint of fresh grout.

28 percent of the compressive strength obtained on the restrained specimens (6750 psi). The low strength of the unrestrained specimens is due for the most part to the free expansion of the grout which in this case was 10.9 percent at three hours.

The data of Fig. 5 clearly indicates that compressive strength tests conducted on unrestrained grout specimens are of no practical value.

GROUTING METHODS

Because of possible corrosion damage to post-tensioning steel, grouting should be performed as soon as is practicable after stress is applied to the cable.

Grout holes at both ends of a duct should be provided with a ¼-in. or preferably ⅜-in. pipe connection. These connections, equipped with a stopcock or plug valve should be tight so that pressures as great as 100 psi may be applied without loss of air or water. For long members of a continuous structure, it is desirable to have vents at high points of the duct for bleeding of air. If the walls of the duct to be grouted are the concrete of the member itself, the duct should be filled with water under pressure for an hour or more before the time of grouting.

Prior to grouting, the duct should be flushed out with water and compressed air, to remove any loose material which might obstruct the free flow of grout.

The grout should be injected by the use of a grout pump of either the piston-displacement type or the

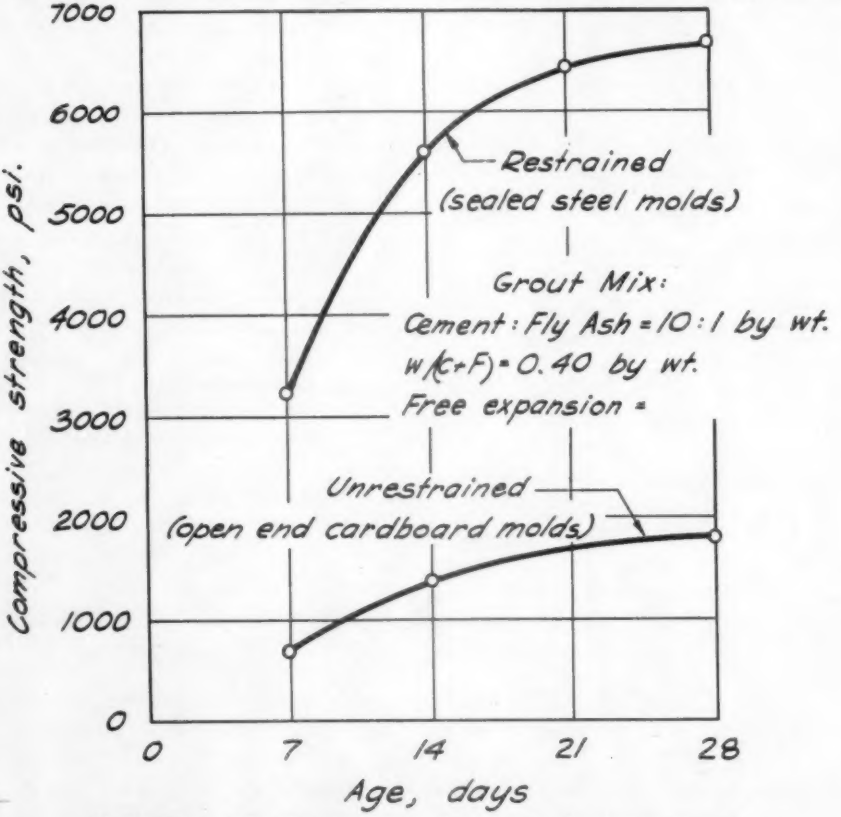

Fig. 5—Effect of Restraint on compressive strength of neat-cement grout.

screw or impeller type. The grout should be pumped continuously at a slow rate until the cavity is filled. The use of a pressure pot as a device for injecting the grout is not advisable since control of the rate of grouting is difficult, and there is always the possibility of slugs of air being forced into the cavity. After grout appears at the vent openings of the duct, grouting should continue (wasting excess grout) until it is clear that all entrapped air has been removed, and the duct is completely filled with grout of good quality. If there are several vent openings, they should be progressively closed in the direction of grout flow. To insure complete filling of a duct, it is also important that it be free of abrupt changes in cross sections, and that there be no interconnection between one duct and another.

As soon as the duct is completely filled with grout, the valve at the outlet end should be closed. A grout pressure as high as 100 psi should then be applied and held for about one minute after which the valve at the near end should be closed. All valves of the duct should be kept closed and in place until final setting of the grout has taken place.

SUMMARY

1. Ducts of post-tensioned concrete members should be grouted after tensioning to protect the steel from corrosion and to provide bond between the tendons and the concrete.

2. Current practice in prestressed concrete pays very little attention to, or is ignorant of, proper grouting procedures and several failures due to poor grouting have been reported.

3. Grouts for post-tensioned concrete members should be composed of cementing material, water a chemical admixture, and sometimes fine sand. The cementing material may be either ordinary portland cement, a high-early strength cement, a portland blast-furnace cement, a portland pozzolan cement, or a portland cement with a finely divided active mineral admixture (pozzolans, natural cement, or granulated blast-furnace slag).

4. Whether a neat cement grout or a sand-cement grout should be used depends on the type and size of cable and size of the tendons. Cables with large void spaces (Magnel cable) will permit the use of sand-cement grouts whereas those having small void spaces (Strescon cable) require the use of neat cement grouts.

5. The chemical admixture, usually in powered form, should be designed to produce an expansion of the grout through generation of tiny gas bubbles, to reduce the mixing water requirement, and to retard the rate of early stiffening and the time of setting. The expansion is produced through the use of aluminum powder. A water-reducing retarder is used to reduce the water requirement and provide retardation of stiffening and setting.

6. Cement composition, mix proportions, and type of admixture will influence properties of grouts. Grout materials and grouts should be evaluated prior to use in the field. The sand for the grouts should all pass No. 30 sieve.

7. Grouts for post-tensioning cables of prestressed concrete members should be of as thick consistency as can be readily injected to completely fill the voids within the duct, should be of low water requirement, should be of very low bleeding characteristics, and should expand while in plastic state. The unrestrained expansion of the grout

should be about 10 percent of its volume.

8. As measured by the flow-cone apparatus, a neat cement grout should have a flow factor of 20 to 24 seconds and a sand-cement grout 24 to 28 seconds.

9. Compressive strengths of grouts should be determined only for specimens which were restrained from free expansion during both their setting and hardening period. Compressive strengths of unrestrained specimens are not applicable.

10. Grout pumps should be employed for the grouting of post-tensioning units. The ducts should be flushed out prior to grouting, and care should be taken to completely fill the void space. Immediately after grouting the valves of the duct should be closed to maintain the grout under restraint during both setting and hardening.

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Recent British Research On Prestressed Concrete

by **A. R. Collins***

Presented at the Prestressed Concrete Institute Convention
September, 1960

INTRODUCTION

I Research on Structural Problems

(a) Beams subjected to combined bending and torsion
(b) Composite beams of prestressed precast and in situ concrete
(c) Transmission length of stranded-wire tendons
(d) Stress distribution in the anchorage region of prestressed members

II Research on Fire Resistance

(a) Effect of high temperatures on concrete and steel
(b) Fire resistance of large post-tensioned beams
(c) Fire resistance of pre-tensioned floor units
(d) Experience in practice

In Britain, as in most countries, there is no one place for research on prestressed concrete. The work is done as a part of the programme of many organizations concerned with research in the construction industry, including government research establishments, universities, co-operative industrial organizations and individual companies. The most important government research organizations working in this field are the Building Research Station and the Road Research Laboratory, both operated by the Department of Scientific and Industrial Research (DSIR), and the Joint Fire Research Organization operated by the DSIR in conjunction with the fire insurance companies. All of these undertake research on prestressed concrete each specializing in its own field. Of the universities those most concerned with prestressed concrete are Aberdeen, Cambridge, Durham, Glasgow, Leeds, London, Manchester, Sheffield, Southampton and Wales. The co-operative industrial organizations most closely involved are the Cement and Concrete Association (almost exactly equivalent to the Portland Cement Association in the USA) and the Prestressed Concrete Development Group, the latter, however, does not undertake research itself but it does on occasion act as sponsor. Very little research is carried out by individual companies, but a number of them naturally undertake development work on their products and conduct experiments accordingly. They and the Government construction agencies (chiefly the Ministries of Works and Transport), together with public utility organizations, co-operate in many ways with those doing research.

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Prestressed concrete was first used in Britain in 1939. By that time Freyssinet had developed the process to the point of practicability, but there were clearly many problems which research alone could solve. Because of the war, however, nothing could be done immediately and research on the subject has been in progress for less than fifteen years.

In terms of the history of the construction industry, therefore, all our research on prestressed concrete could legitimately be included under the title chosen for this paper. However, limitation of space makes a highly selective approach essential and in the first part of the paper, dealing with structural problems, the work described has been restricted to that currently in progress at the Research Station of the Cement and Concrete Association.

It is understood that the Members of the Prestressed Concrete Institute have at present a special interest in fire research, and as the Cement and Concrete Association does not operate in this field, some account of the work on prestressed concrete of the Fire Research Organization has therefore been given in the second part of the paper. It must be emphasized, however, that except through bonds of common interest the Author and the Cement and Concrete Association are in no way connected with the Fire Research Organization.

I Research on Structural Problems

(a) *Beams subjected to combined bending and torsion*

Because of a lack of information on the behaviour of structural concrete members under combined bending and torsion, the practice has grown up of designing structures to avoid all but very small twisting moments. This may not always be economically or aesthetically satisfactory and an investigation is now in progress to provide data that it is hoped will enable both elastic and ultimate load analyses to be made of structural members subjected to combined bending and torsion (1).

The apparatus shown in Fig. 1 has been designed to accommodate beams up to about 10 ft. long. The loads causing the bending moment are applied at the third points and torsion applied by means of a short cross member fixed to each end of the test beam connected together by a beam running diagonally under the test beam. The end bearings on the test beam are designed to allow freedom for twisting as well as bending. Loading is applied by hydraulic jacks and measured by pressure capsules. Reactions are taken to the testing floor to which the apparatus is fixed.

Three series of tests have so far been undertaken. The first was on rectangular beams with a section 8 in. deep by 5 in. wide, prestressed with 5 wires each 0.276 in. in diameter with an eccentricity of 1.5 in. The second was made on I-beams with a section 8 in .deep by 5 in. wide with a web 1½ in. thick and flanges having an average thickness of 1½ in. The beams were prestressed with six wires each 0.2 in. in diameter, four in the lower flange and two in the web. The prestressing was designed to give an effective eccentricity of 0.2 in. The third series was made on T-beams with an overall depth of 8 in., a flange width of 10 in. and web and flange thicknesses of 2 in. The prestressing was obtained with two wires 0.276 in. in diameter 2½ in. up from the bottom and one wire 0.2 in. in diam-

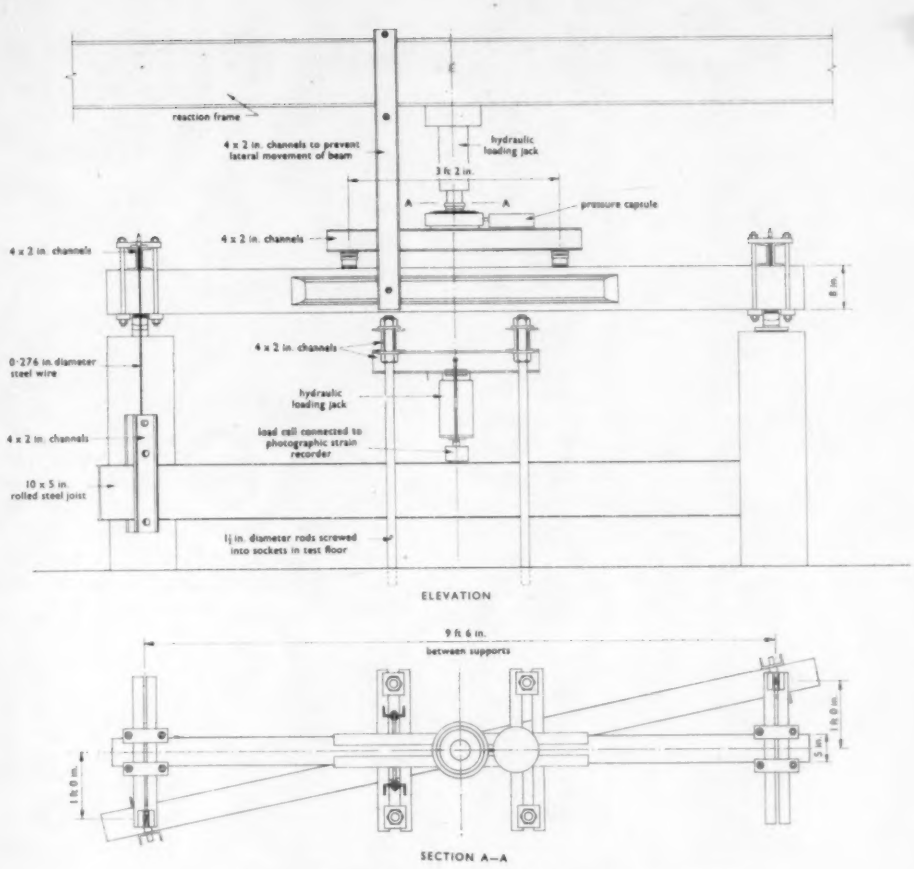

reaction frame
4 x 2 in. channels to prevent lateral movement of beam
hydraulic loading jack
3 ft. 2 in.
A
A
pressure capsule
4 x 2 in. channels
4 x 2 in. channels
4 x 2 in. channels
0·276 in. diameter steel wire
hydraulic loading jack
4 x 2 in. channels
load cell connected to photographic strain recorder
10 x 5 in. rolled steel joist
1¼ in. diameter rods screwed into sockets in test floor
8 in.
ELEVATION
9 ft. 6 in.
between supports
1 ft. 0 in.
1 ft. 0 in.
5 in.
SECTION A—A

Fig. 1: Apparatus for testing combined bending and torsion.

eter 4½ in. from the bottom. The position of the wires in all the beams was checked after testing and any variations in eccentricity were allowed for in the calculations. The prestressing wires were passed through individual small ducts (about 0.3 in. diameter) and post-tensioned when the concrete had a cube strength of about 6,000 per sq. in. They were afterwards grouted with neat cement paste.

All the I- and T-beams were made rectangular at each end to facilitate the fixing of the torsion apparatus and mild steel reinforcement was used to prevent any torsion failure in the ends.

Resistance strain gauges were fixed longitudinally on the flanges and in a delta arrangement on the web, all readings being recorded photographically on an automatic Wheatstone bridge. Rotations were measured by an arrangement of mirrors and by a mechanical gauge depending on the extension of a thin steel strip 8 in. long fixed to two steel posts cast vertically one in each side of the top flange.

In a typical test the first stage consisted of a determination of the elastic modulus by the application of a bending load of 2 tons applied twice in increments. Then at a given

VOLUME TIGHTLY

Table I: Results of tests on Rectangular, "I" and "T" beams under combined bending and torsion

(1) Rectangular beams

Beam number	Bending moment lb-in.	Ultimate twisting moment lb-in.	Cube strength lb/in.^{2}	Flexural strength (2) lb/in.^{2}	Cylinder splitting strength (3) lb/in.^{2}	Age at test days
1	212,000[1]	0	6,720	700	—	30
2	0	44,940	7,000	740	—	35
3	53,400	48,740	6,520	700	—	30
4	106,800	41,740	6,680	720	—	30
5	159,600	36,190	6,790	720	—	30
6	0	43,100	6,520	685	—	30
7	26,600	45,190	6,630	700	—	30
8	80,200	44,240	6,550	665	—	30
9	133,800	36,790	6,760	645	—	30
10	187,200	24,900	6,340	670	—	30
11	26,600	43,700	6,280	680	—	30
12	47,600	46,240	6,200	710	—	30
13	204,000[1]	0	6,270	700	—	30
14	70,200	50,940	6,170	715	—	30
(2) I-beams						
1	21,700	21,200	6,660	630	640	31
2	28,100	18,600	6,080	620	450	30
3	28,100	22,200	6,640	610	490	31
4	49,400	20,900	7,680	580	450	28
5	49,400	21,000	7,750	600	500	30
6	70,700	20,700	7,510	600	500	28
7	92,000	19,100	7,620	680	540	30
8	92,000	22,000	7,090	610	440	30
9	113,200	20,400	7,170	630	540	30
10	134,500	20,100	6,830	570	500	30
11	134,500	22,700	7,020	640	470	30
12	177,000	21,600	7,070	660	420	30
13	177,000	18,200	6,690	610	450	28
14	177,000	19,100	7,780	570	480	30
15	217,500[1]	—	6,250	600	460	29
16	219,600[1]	—	7,930	650	490	29
(3) T-beams						
T 1	119,000[1]	—	6,530	625	565	32
T 2	151,100[1]	—	7,340	700	540	29
T 4	0	23,000	9,040	880	550	71
T 5	0	17,000	9,660	650	595	57
T 6	24,100	23,000	9,880	705	600	57
T 7	38,000	25,000	7,810	1080	610	65
T 8	57,000	26,060	8,920	785	610	65
T 9	0	13,860	7,630	815	570	67
T.0	76,000	28,710	9,000	835	605	77
T11	95,000	25,660	9,210	1165	625	74
T12	104,500	20,960	9,120	950	654	74
T13	113,000	27,990	10,680	845	690	80

(1) Ultimate bending moment with no torsion.
(2) Modulus of rupture of 4 in. x 4 in. x 20 in. beams.
(3) Tensile strength of 6 in. x 12 in. cylinders tested on side.

Fig. torsi No.

GHTLY BOUND

Fig. 2: T-beam after test in combined bending and torsion showing compression failure in top flange (beam No. T11, Table 1)

bending load twisting moments first of 8,000 lb-in. and then 12,000 lb-in. were applied twice each in increments. Finally the beam was twisted to failure.

All the beams in each series were of about the same strength and each was tested to failure by twisting under a different value of bending moment ranging from zero up to about 80 per cent of the ultimate moment in pure bending.

Some of the results are given in Table 1 and they show clearly that the ultimate twisting moment is not greatly affected by the applied bending moment. It is interesting to note that this remained true even when the applied bending moment was sufficient to cause severe cracking before the application of the twisting moment. A possible explanation of this phenomenon is that the existence of high bending compressive stresses allowed the compression zone to carry a very high shear force without the development of tensile stress.

A finite difference solution of the governing differential equation for the I-beams, carried out by electronic computation, gave a good estimate of the elastic behaviour of the beams and an assumption of full plasticity at failure also gave a good estimate of the ultimate torsion moment.

The work on the T-beams is still in progress and the results have not yet been fully analysed. Photographs of a typical T-beam after test (Beam No. T11 in Table 1) are shown in Figs. 2 and 3.

(b) *Composite beams of prestressed, precast and in situ concrete*

A number of different systems of bridge and floor construction involve the use of precast prestressed concrete units combined with in situ reinforced concrete to form composite beams. This combination of somewhat dissimilar components into one structural unit is very useful as a means of construction but it does present a number of questions regarding the behaviour of the unit under load.

Effects due to differential shrinkage and the behaviour of concrete in conditions of restrained tension are of some importance. And designers need to be able to estimate the effective width of an in situ flange in a composite T-beam and calculate cracking moments.

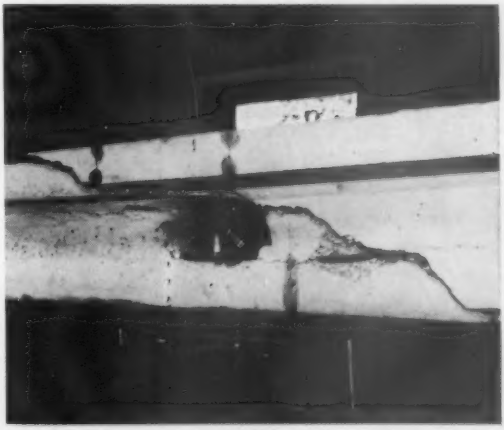


Fig. 3: T-beam after combined bending and torsion test (Beam No. T11, Table 1)

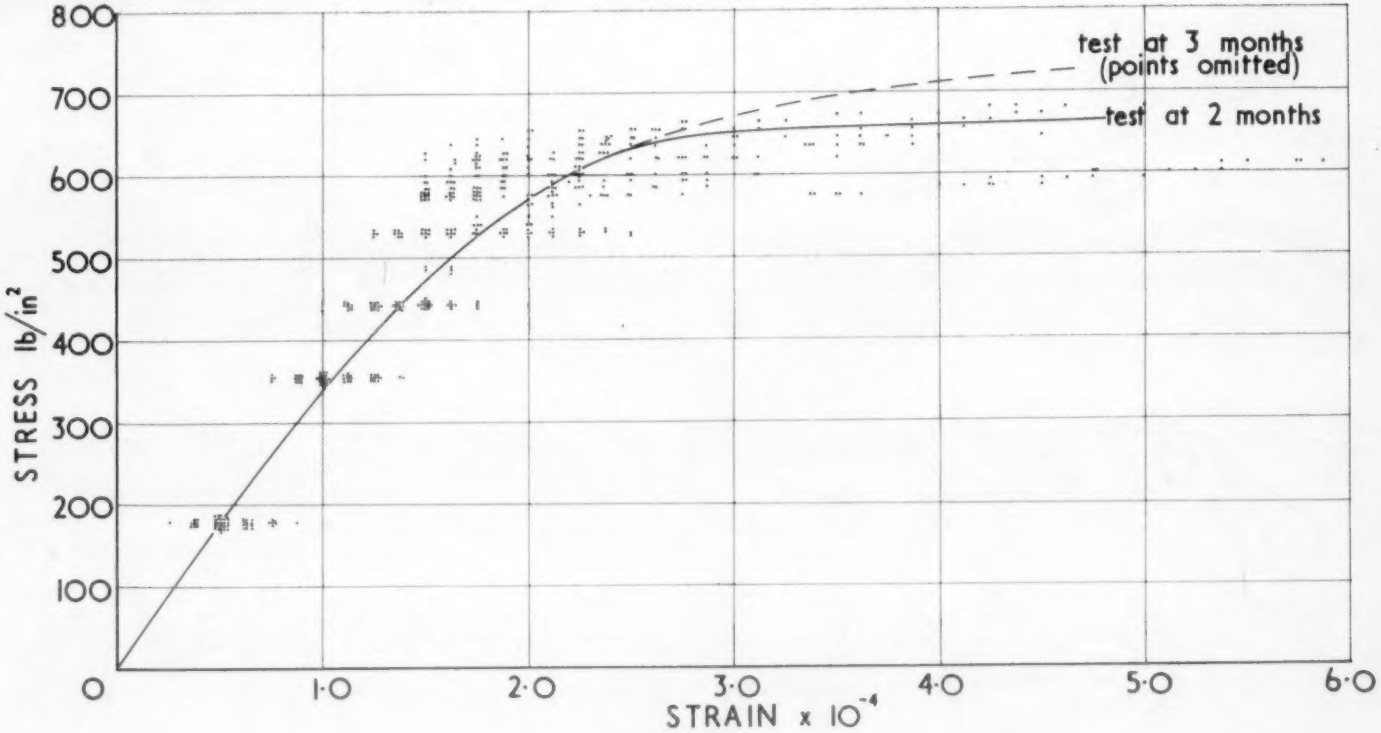
An investigation designed to give some information on these factors is now in progress. The first phase consists of tests in bending of rectangular beams 10 ft. long. These were made by casting in situ concrete on a precast prestressed plank having a width equal to the full width of the beam. The beams were of two sizes having overall sections of 8 in. x 5 in. and 6 in. x 5 in. respectively. These dimensions include the prestressed portions which were all 5 in. wide and 2 in. deep and prestressed uniformly to about 2,000 lb/in.^{2} by means of five wires 0.2 in. in diameter. The concrete in the prestressed part had a 28-day cube strength of about 10,000 lb/in^2 and the in situ concrete forming the rest of the beam was made in three qualities having 28-day cube strengths of 4,000, 5,500 and 8,000 lb/in.^{2} respectively. The top surface of the plank was provided with a normal trowelled finish but two shear connectors, each consisting of a ⅜ in. diameter mild steel bar bent in the form of a Z, were placed at each end. Shrinkage measurements were made on both parts of the beams from the time of casting until testing in flexure. During testing strain measurements were made on both parts of the beam over its whole length and values of the cracking moments and the appearance and growth of cracks were recorded.

The control specimens used consisted of 4-in. cubes, shrinkage prisms (3 in. x 3 in. x 10 in.); beams for flexural strength (4 in. x 4 in. x 20 in.); and cylinders (6 in. x 12 in.) for side testing to give an estimate of the tensile strength of the concrete. Measurements of the ultimate tensile strain were made on the flexural test beams, the cylinders and on the composite beams.

The investigation is by no means complete but already some interesting points have arisen. In the tests on the control cylinders tested on the side by splitting to produce a tension failure, the measurements of strain taken up to the point of failure indicated that quite a large plastic tensile strain could occur without visible cracking. The load-strain relation shown in Fig. 4 remained approximately linear up to a strain of 1.5×10^{-4} but the rate of increase of strain then became very much greater and strains of the order of 4 to 5×10^{-4} occurred just prior to failure. On the small flexural test beams the ultimate strain was not more than about 2×10^{-4}, the difference being no doubt due to the difference in the degree of restraint applied to the tension zone in the two tests.

Evidence of high tensile strains was also obtained from the tests on the composite beams, for cracks did not appear first in the in situ concrete alone. They appeared either in the underside of the prestressed plank or simultaneously in both the in situ and prestressed concrete at the interface between the two parts of the beam. Although the modulus of elasticity of the in situ concrete was lower than that of the prestressed concrete, the strain at the interface at the time the first crack occurred was substantially higher than would be expected in an ordinary reinforced concrete beam.

The diagram shown in Fig. 5 gives the surface strains measured on a typical beam at the cracking load (0.77 of the ultimate load) and at 0.90 of the ultimate load. The figures marked on the diagram give the strain in units of 10^{-4}, the curved "horizontal" lines show strain contours (the dashed line being the neutral axis) and the "vertical" lines indicate the strain across the section.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STRESS lb/in²
STRAIN x 10⁻⁴
test at 3 months
(points omitted)
test at 2 months

Fig. 4

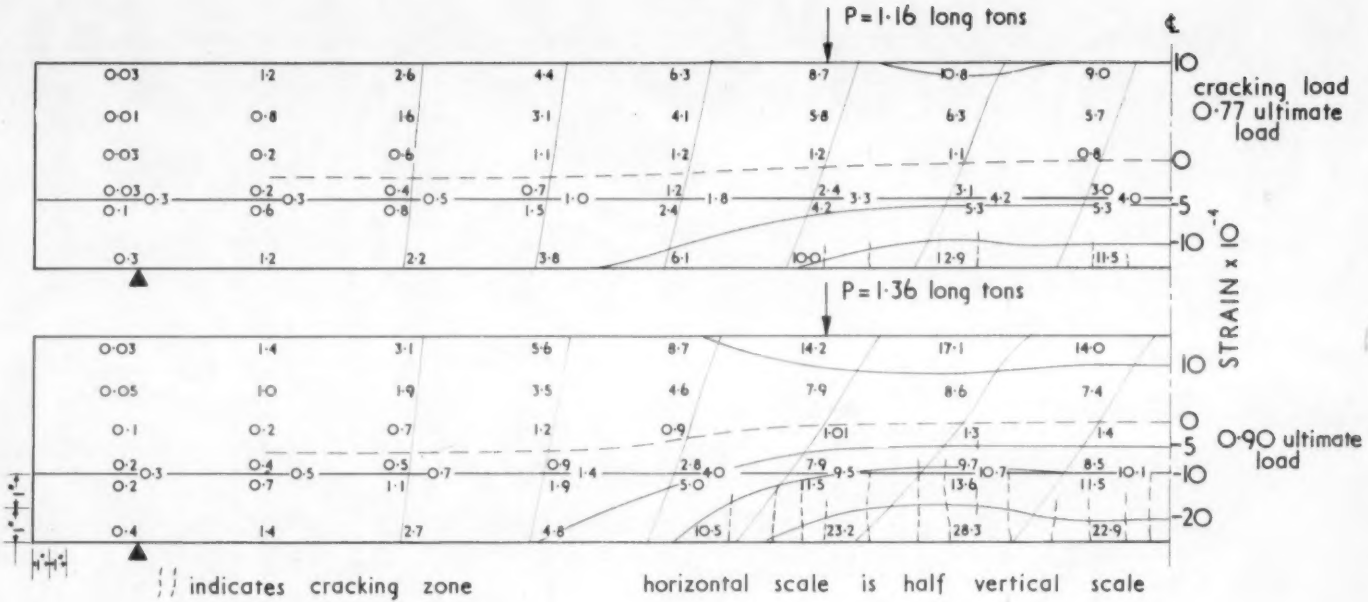

P = 1·16 long tons
P = 1·36 long tons
cracking load
0·77 ultimate load
0·90 ultimate load
STRAIN x 10⁻⁴
indicates cracking zone
horizontal scale is half vertical scale

Fig. 5

The straight horizontal line 2 in. from the lower edge represents the boundary between the prestressed and in situ parts of the beam, and the figures along this line give the values of strain measured at the interface between points fixed alternately on the prestressed and in situ portions.

An interesting point that has emerged from the tests is that, although the surface of the prestressed planks on which the in situ concrete was cast was almost smooth, no slipping occurred in any of the specimens and cracks in the in situ concrete generally coincided with those in the prestressed plank.

The contrast of this result with experience of the difficulties in obtaining bond for floor toppings indicates a need for more research on the nature of bond between two layers of concrete.

Tests are now in progress with beams of T-section and it is hoped from these to obtain data on the effective width of flange obtainable with composite construction.

(c) *Transmission length of stranded-wire tendons*

When strand is used for pretensioning, the load in the inner wires has to be transferred to the concrete by friction through the outer wires which are themselves unlikely to be in contact with the concrete for much more than half their circumference. There naturally arises, therefore, the question whether the full strength of the strand can be utilized.

Comparative tests on beams prestressed with strand and spaced individual straight wires have, therefore, been made in order to determine the transmission length obtained. Three different strands were used; one with an overall diameter of ⅜ in. composed of seven wires (6 on one) designed for an initial pestressing force of 14,400 lb, one of ½ in. diameter with seven wires (6 on one) designed for a force of 25,500 lb. and one of 0.7 in. diameter with nineteen wires (6 and 6 on 6 on one) designed for a force of 53-200 lb.

The beams tested contained either one 0.7 in., two ½ in. or four ⅜ in. strands. They were 11 ft. long and had a rectangular section of 6 in. by 9 in. Comparison beams of the same dimensions were made with eleven single wires of 0.2 in. diameter, and all beams had applied to them a similar total prestressing force giving extreme upper and lower fibre stresses in the concrete of 0 and 2,000 lb. per sq. in. respectively. In each instance the tendons were surrounded over the end 18 in. of each beam by a helix 4 in. in diameter and 1 in. pitch composed of ⅜ in. diameter mild steel reinforcing. All the tendons were used in the condition in which they were supplied by the makers—clean and unrusted. The concrete had a cube strength of from 4,000 to 7,000 lb. per sq. in. at the time of transfer. Transmission length was calculated from measurements of the build-up of strain over the end portions of each beam made on 32 overlapping 8 in. gauge lengths.

A summary of the results obtained is shown in Table 2, the transmission length quoted being the distance from the end of the beam to the section at which the full prestressing stress was developed in the concrete. The results were found to be unaffected by variations in the strength of concrete between the limits of 4,000 and 7,000 lb per sq. in. included in the tests.

Table 2: Transmission lengths obtained with tendons of strand and individual wires

Tendon	Transmission length
	Inches
3/8 in. overall diameter 7-wire strand	7 to 8
1/2 in. overall diameter 7-wire strand	11 to 14
0.7 in. overall diameter 19-wire strand	18 to 22
0.2 in. individual wires	15 to 27

Tests are now in progress to determine the behaviour of the beams at failure. The beams were supported 2 in. from the end and the loads applied at points 15 in. and 33 in. from the ends. No bond failure has yet occurred, although cracks developed 15 to 20 in. from the ends.

Other types of failure are being investigated using T-beams with low percentages of steel.

(d) *Stress distribution in the anchorage region of prestressed members*

The Anchorage of a post-tensioned tendon applies a load of considerable magnitude over a comparatively small area of concrete and substantial compressive and tensile stresses may therefore develop in the anchorage region. A theoretical study of the stress distribution in the anchorage region presents several intractable problems, and the only theories that it has been possible to produce have of necessity been based on simplifying assumptions.

An experimental study also presents a serious problem for it is complicated by the need to deduce stresses from measurements of strain and by the difficulty of measuring internal strains without disturbing the stress distribution. Simplification is therefore again necessary and in the investigation now in progress in the Laboratory of the C & C A the work is being undertaken in a number of stages (2)(3). The first consisted of measurements of the surface strains in prisms of concrete made to simulate the end region of a prestressed-member. The anchorages used were those most commonly employed in Britain and the prisms all contained holes to represent the normal prestressing duct. A photograph of a typical specimen under test is shown in Fig. 6, which in this instance is being tested with the load applied over the full cross-sectional area. Strain measurements were made with a simple mechanical gauge 2 in. long operating between pairs of small steel discs each containing a

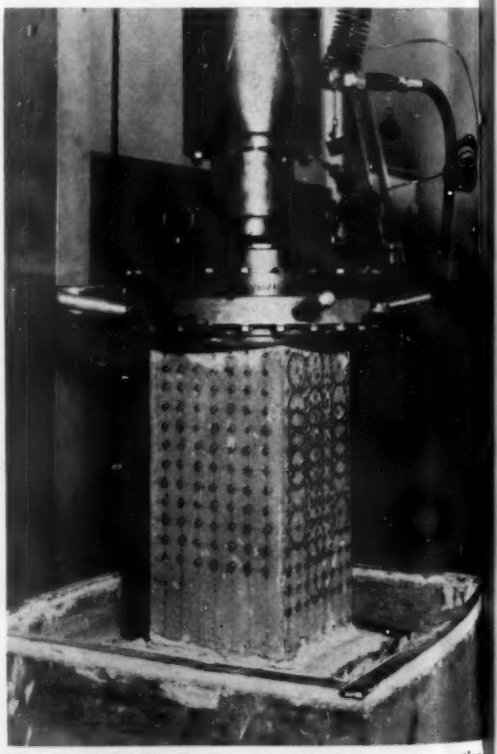


Fig. 6: Stress-distribution investigation; square-section specimen under test.

center hole for the peg of the gauge. Various patterns of gauge points were applied to the specimens as shown in the photograph.

In the first series of tests the prisms were of square section and three factors were examined: the effect of varying the ratio of loaded area to the total area of the section of the concrete, the differences due to embedded and external anchorages and the effect of the amount and type of mild steel used to reinforce the block.

The results were expressed in terms of the ratio $\frac{a_1}{a}$ where $2a_1$ is the length of side of the square loaded area or the square root of the area of a circular loaded area and $2a$ the length of side of the section of the prism. They showed that the position of maximum and zero stress when expressed as a function of a were almost independent of the ratio $\frac{a_1}{a}$, zero stress being about $0.15a$ and maximum stress about $0.5a$ from the loaded face. All the calculations based on the theories of Magnel, Guyon, Bleich and Bleich-Sievers gave values ranging from about 1.2 to 3 times the experimental figures.

Differences of similar magnitude were also found between the theoretical stresses and the experimental results. These are shown in Fig. 7 in which tensile stress is represented by the ratio $\frac{f_t}{f_c}$ in which f_t is the maximum tensile stress and f_c is a uniform compressive stress calculated by dividing the total load by the cross-sectional area of the prism. It will be seen that the stresses obtained from the tests are all higher than those deduced from all the theories and that some of the differences are as great as three to one. The differences between the effects of external and embedded anchorage appeared to be slight.

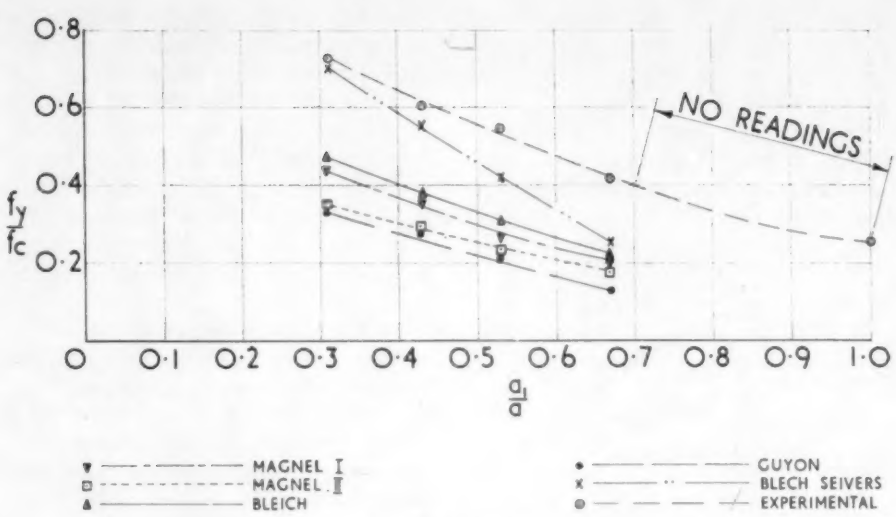
As would be expected mild steel in the anchorage region did not affect the incidence of cracking but it was shown that there was a limit to the amount of steel above which increasing amounts ceased to have any significant effect on the ultimate load.

In the second series of tests the square prisms were replaced by I-section specimens as shown in Fig. 8, and in later tests multiple load points were used to represent groups of anchorages. The results have not yet been fully analysed but the general trend appears to be similar to that shown by the tests on square prisms. Tests are now being planned in which the strains within the specimens will be measured by means of embedded gauges.

II Research on Fire Resistance

The Fire Research Organization studies many aspects of fire protection, including the causes, spreading and extinction of fires as well as the fire hazard of building materials, the behaviour of structural units and the requirements for buildings. Descriptions of its work are given in the annual reports of the Organization and in various other publications (see refs 4 to 11) and reference should be made to these for further information on what follows.

Research on matters concerning the fire resistance of prestressed concrete have included studies on the effect of high temperatures on concrete and steel, measurements of the temperatures reached beneath the surface of concrete exposed to fire, and the testing of prestressed beams and floors.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MAGNEL I
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BLEICH
GUYON
BLECH SEIVERS
EXPERIMENTAL

Fig. 7

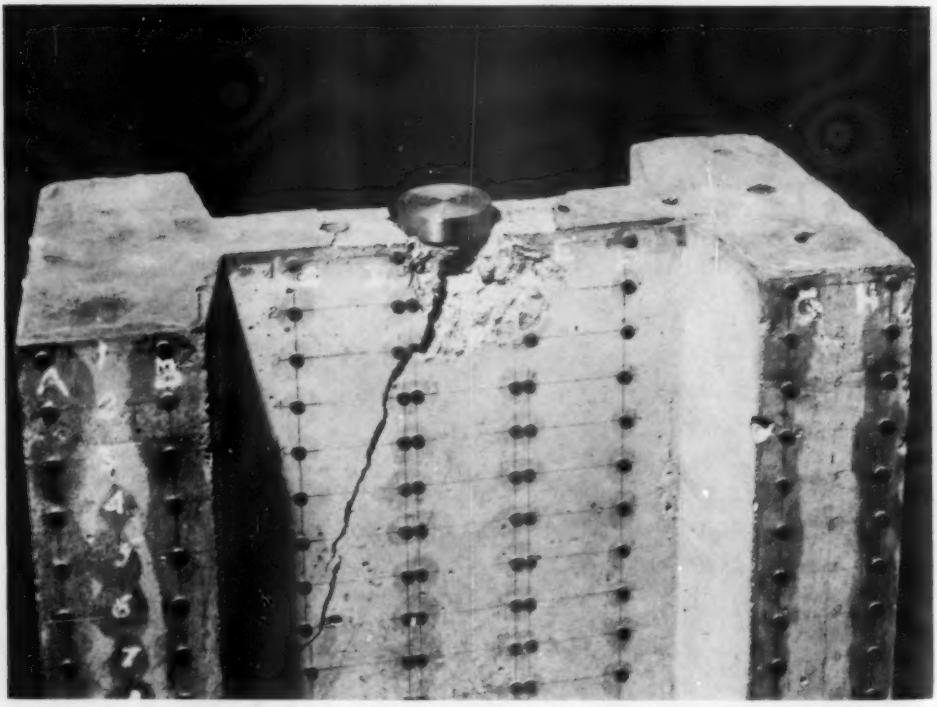

Fig. 8

(a) *Effect of high temperatures on concrete and steel*

The work on the properties of concrete exposed to high temperatures has been described by Malhotra[5] and summarized by Hill[6]. The results have shown that as the temperature of concrete is raised to about 300°C the normal thermal expansion at lower temperatures is at first offset and then exceeded by dehydration shrinkage in the cement paste. At this stage continued expansion of the aggregate particles sets up internal stresses and with the lowering of the strength of the paste through dehydration the concrete ultimately disintegrates. Serious loss in strength occurs at temperatures of about 600°C and most concretes disintegrate at about 900° C, both temperatures naturally depending on various factors, including the aggregate used.

Among the conclusions reached by Malhotra for concrete made from ordinary Portland cement with fine and coarse flint aggregates are:—

(1) The effect of temperature on the crushing strength of concrete is independent of the water/cement ratio within the range normally used in its manufacture.

(2) The aggregate/cement ratio has a significant effect on the strength of concrete exposed to high temperature, the proportional reduction being smaller for lean mixes than for rich mixes.

(3) Concrete under a compressive stress of the order of its design stress has a smaller proportional decrease in strength than if the stress were absent.

(4) The residual strength of heated concrete shows still further reduction in strength on cooling, being approximately 20 per cent less than the corresponding hot strength in the temperature range 200 to 450° C for mixes of 1:9 and 1:6 proportions by weight.

Changes in physical properties are accompanied by changes in colour (7). This is a useful phenomenon for it provides a practical guide to the extent to which the concrete has been affected and indicates the parts of a fire-damaged building requiring repair.

The properties of high-tensile cold-drawn steel commonly used in prestressed concrete are also severely degraded by high temperatures. It has been found, for example, that the modulus of elasticity is reduced by about 20 per cent at 300°C and the strength is halved when the temperature reaches 400°C. It is clear, therefore, that prestressed concrete may be in imminent danger of collapse when the temperature of the steel reaches 400°C (6).

It is well appreciated in Britain, as in America, that fire resistance is a relative property than can strictly be applied only to complete systems of construction. It is also a function of time and the occurrence in a fire of temperatures that would destroy the materials of a structure does not mean that there is any danger of immediate failure. Apart from the effect of finishes in delaying temperature rise in structural units, concrete is itself not a very good conductor of heat. A dangerously high temperature may therefore not be developed in the interior of a concrete member for some considerable time even when the surface is exposed to a severe fire. Measurements of the temperature in the interior of a concrete wall, the surface of which is exposed to the standard temperature-time heating regime (see Fig. 9), have been made and some of the results are given in Fig. 10.

(b) *Fire resistance of large post-tensioned beams*

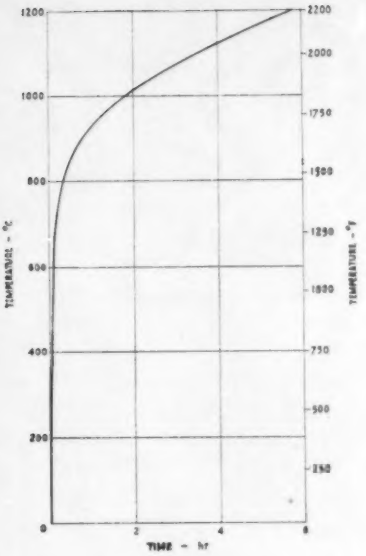

Fig. 9

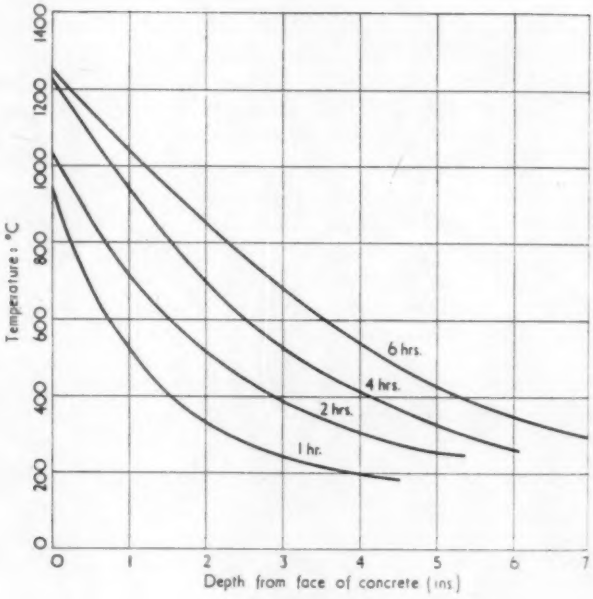

Fig. 10

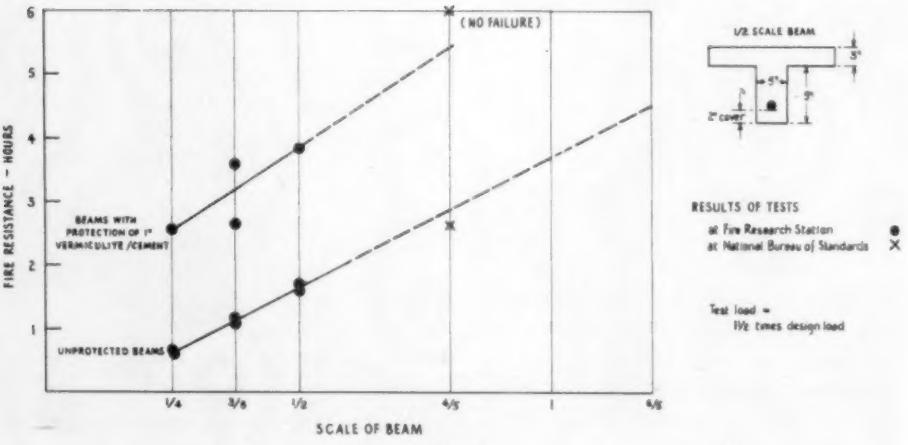

Fig. 11

In Britain, as in America, tests on structural units are made with specimens in the form of beams, columns, walls and floors; beams and columns are generally at least 10 ft long and walls and floors 10 ft square. Heat is applied by gas furnaces designed to give the same standard time-temperature curve as is used in the USA (8) and shown in Fig. 9. Loads are applied during tests and observations made continuously. During the tests three critical phases are recognized:

(1) when the remote side of a wall or floor becomes hot enough to ignite inflammable material in contact with it;
(2) when cracks or holes form that allow hot gases to pass through a wall or floor and so spread the fire;
(3) when the specimen collapses.

The fire resistance of any structural unit is taken as the time from the start of the test to the occurrence of one of the above criteria of failure and is generally given to the next lower of one of the following times:—½, 1, 2, 4 or 6 hours (9).

In Britain an early major use of prestressed concrete in conditions where there was a high fire hazard was in the construction of warehouse floors. The beams used in these floors were however too large for the furnace then available. Tests were therefore planned with the double objective of determining the resistance of the beams in question and of establishing the accuracy of model tests in this field. A series of beams of varying scale, ranging from one-quarter to four-fifths full size, were therefore prepared, the largest one being designed for testing in the furnace at the Bureau of Standards in Washington, which at the time was the largest in existence. (The Fire Research Organization now has a furnace for beams of 24-ft span.)

All the tests were made with the standard procedure and rate of heating and the results obtained are shown in Fig. 11. There is clearly a good degree of correlation between scale and fire resistance and between the fire tests made in Britain with the one made in the USA.

With the installation of a larger beam furnace at the Fire Research Station a further series of beams has been tested. Complete results have not, however, yet been made available but the following tentative conclusions have been published by Hill & Ashton and have formed the basis for the fire resistance requirements in the new British Code of Practice for Prestressed Concrete (10).

1. Explosive spalling of unprotected beams, having no part of the section less than 2 in. is unlikely.

2. Failure of a beam can be regarded as imminent when the mean cable temperature at any section reaches 400°C.

3. A fire resistance of one hour can be obtained with a concrete cover to the cable of 1½ in., and a fire resistance of four hours with a cover of at least 4 in. In order to obtain the four-hour period it will be necessary to incorporate mesh reinforcement round the prestressing steel to retain the cover in place.

4. Collapse is likely to be gradual. Warning of failure is given by the development of a marked sag, which may increase visibly just before the end.

5. There is little difference in performance between beams of rectangular section and beams of I-section designed for the same load and hav-

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Use of Plastiment Retarding Densifier increased the workability of the Ridge-

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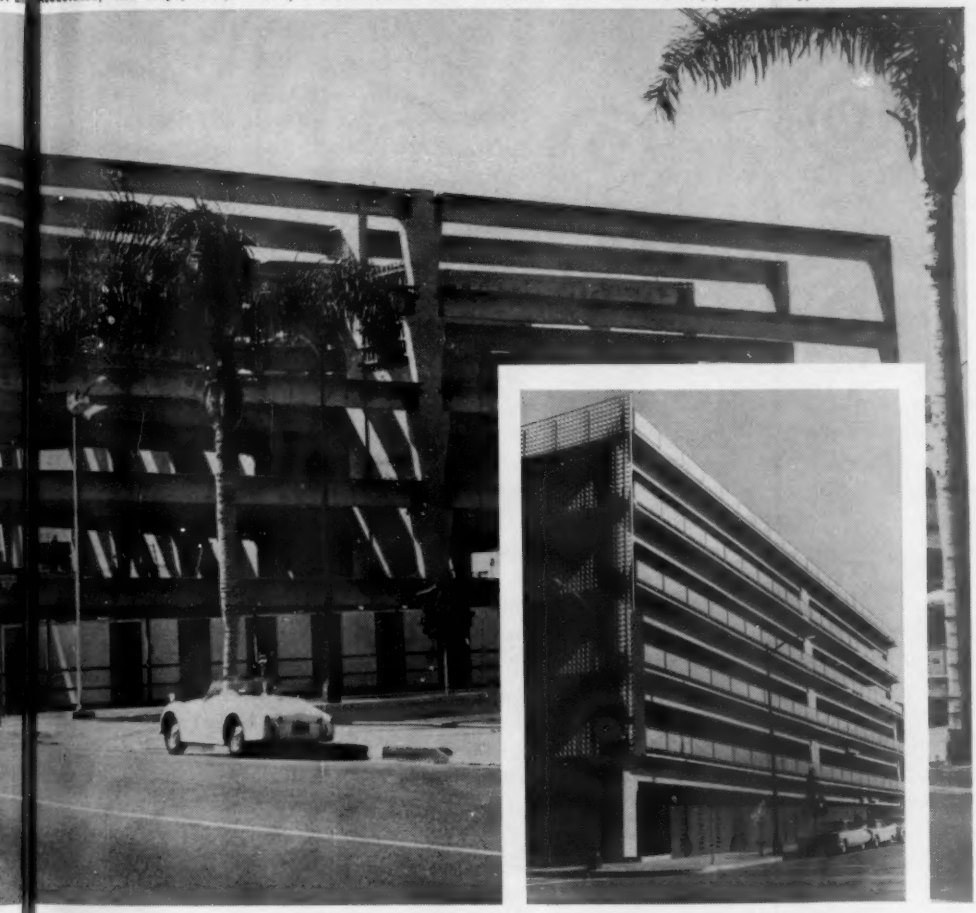
...n Becket & Associates, Los ...ssociates, Van Nuys, Cal.; **General Contractors:** C. L. Peck, Los Angeles, Cal.; Ellis E. White Co., Los Angeles, Cal.; **Precast Columns & T's:** Wailes Precast Concrete Corp., Sun Valley, Cal.



lite lightweight concrete thereby assuring a smooth, clean appearance to the exposed concrete members. Using high early cement, strengths averaged over 3,500 psi in 16 hours in the precast elements. Slab concrete placed at the site reached 3,500 psi in 2 days. 28-day strengths averaged 6,500 psi.

Mix contained 7-1/2 bags of cement per cubic yard and 2 fluid ounces of Plastiment per sack of cement.

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ing the same cover to the cable.

6. A beam longitudinally restrained may have a lower fire resistance than when simply supported.

7. Even after heating for a period of about one-half of that necessary to cause collapse, a beam will show an appreciable loss of prestress on cooling. A marked permanent deection will be present after removal of the imposed load, but the ultimate strength may still be a high proportion of its original value.

8. Protection may also be afforded by the addition of surface finishes adequately fixed to the concrete. Thicknesses of surface finishes to give an increase in fire resistance of about two hours are given below:

Material	Thickness
Vermiculite concrete slabs placed as a lining to the mould ...	1 in.
Vermiculite gypsum plaster ...	⅞ in.
Sprayed asbestos	¾ in.

(c) *Fire resistance of pre-tensioned floor units*

Precast pre-tensioned flooring units have been one of the major uses of prestressed concrete in Britain and it has naturally been a matter of urgency to determine the fire rating of each type of unit. A series of *ad hoc* tests has therefore been made on each unit as it came on the market.

Fire resistance test data for these units and a number of other commercially-produced structural elements of various kinds have recently been collected together in one document and published by the Joint Fire Research Organization (11).

Construction details and results for a selection of these floors have been given by Hill & Ashton (9). These Authors have classified the units into four categories:—

1. Isolated joists requiring flooring of precast concrete or boards, and a continuous ceiling of building boards or plaster on lath.

2. Members of a variety of shapes, spaced apart with hollow infilling blocks of clay or clinker concrete; the top is finished with an in-situ screed and the soffit is plastered.

3. Units placed side by side to form continuous surfaces which are finished on top with a screed and underneath with plaster either applied direct or on metal lath; the units have many forms: T-, U- and box-sections, planks, etc.

4. Composite constructions consisting of beams of heavy section supporting secondary members with in-situ concrete; in a fire the prestressed concrete member is exposed to heat on sides and soffit.

They have also commented on the results as follows:—

The specimens of each type were reasonably consistent in behaviour under the test conditions. Those of type 1 were relatively unaffected until the ceiling fell, but since the joists were of small section their exposure to the furnace resulted in explosive spalling. Floors of type 2 gave good results, since the units were protected on their soffit with plaster, which was provided with a mechanical key by the blocks. Applications of normal plaster remained in place to a large extent throughout the tests and full advantage was obtained from their protective value in preventing spalling of the units and reducing the rate of rise of temperature of the wires. Vermiculite-gypsum plaster provides good thermal protection; it can be applied in thicknesses up to 1 in. without the use of lathing and has high adhesion to concrete.

The units of type 3 did not provide surfaces to which normal plasters adhered well at high temperatures and the behaviour of the

plaster in the tests was variable. Early and extensive falls of plaster might lead to spalling of the exposed concrete with a consequent earlier failure of the floor than if the plaster remained intact. This uncertainty in the behaviour of the plaster makes prediction of the fire resistance of this type of floor difficult. One floor composed of prestressed planks covered with plaster on the under surface did, in fact, suffer from severe spalling due to early failure of the plaster and was prevented from collapsing only by reinforcement in the topping. The value of separate plaster ceilings on metal lathing has, however, been well demonstrated. In one floor an estimated fire resistance of one hour was raised to an actual resistance of four hours by a separate ceiling.

Floors of type 4 included in this series consisted of units of substantial section uniformly prestressed with a large number of wires distributed in rows at different depths. These units did not spall and their resistance to collapse was greater than might be inferred from the depth of concrete cover to the lowest row of wires.

(d) *Experience in practice*

The testing of fire resistance is an expensive and elaborate procedure and it is not surprising that numerous results are not available. The work done in Britain has however covered the essential requirements of knowledge on the subject and enough is now known to give confidence to the designers of all types of building. Fire resistance ratings are incorporated in all the building regulations and bye-laws in force in Britain and the information available has been sufficient to satisfy all bye-law authorities regarding the fire resistance of prestressed concrete. No undue difficulty was experienced in drawing up the new Code of Practice mentioned earlier.

There have been very few severe fires in prestressed concrete buildings, but those that have occurred have given no reason for changing any of the regulations.

ACKNOWLEDGMENTS

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Prestressed Concrete In The Port of New York Authority's Planning

By John M. Kyle*

Presented at the
6th Annual Convention of the Prestressed Concrete Institute
September 28, 1960

For a third of a century the theory of a valid method of prestressing concrete has been known and, at least in the laboratory, has existed in model form. You will remember that it was in 1927 that Professor Freyssinet finally established that mild steel was not suitable for prestressing concrete and that the only method of achieving a permanent compression was by the use of steels having very high yield points as tensioning members.

In the intervening years, giant strides have been made in transforming a laboratory technique into the prestressed industry. In reference however, ours is an infant industry founded on the commonly used techniques of conventional reinforced concrete.

Concrete was of course known to the Romans and is evidenced by such monuments as the Pantheon. The art was then lost for almost seventeen centuries. It was not until 1824 with the development of modern Portland cement that concrete again became a recognized structural material. It was not until 1873 that the theory and practice of reinforced concrete was developed. Today's high strength reinforced concrete is a far cry from the dome of the Pantheon in Rome. Considering the state of the art at the present time, it is probable that prestressed concrete will also undergo some radical changes before completely becoming the durable, low cost, high stress ideal material so desirable in any modern construction.

I use the words, "any modern construction," advisedly. As Chief Engineer of The Port of New York Authority, it has been my privilege to incorporate prestress into the designs of such diversified projects as piers, tunnels, bridges and airports. In the short time since 1954, contractors have in fact installed better than six million dollars worth of prestressed concrete work on Port Authority projects.

The attached map (Sketch #1) indicates the locations of various projects where the Port Authority has employed prestressed elements in its construction program.

A. Hoboken Piers.
B. Port Newark.
C. Brooklyn Piers.
D. Pier 79.
E. Lincoln Tunnel.
F. North Bergen Viaduct.
G. Bridge over Pennsylvania Railroad Yard.
H. George Washington Bridge.
I. New York International Airport.

During this construction, we encountered some problems and developed some solutions which I

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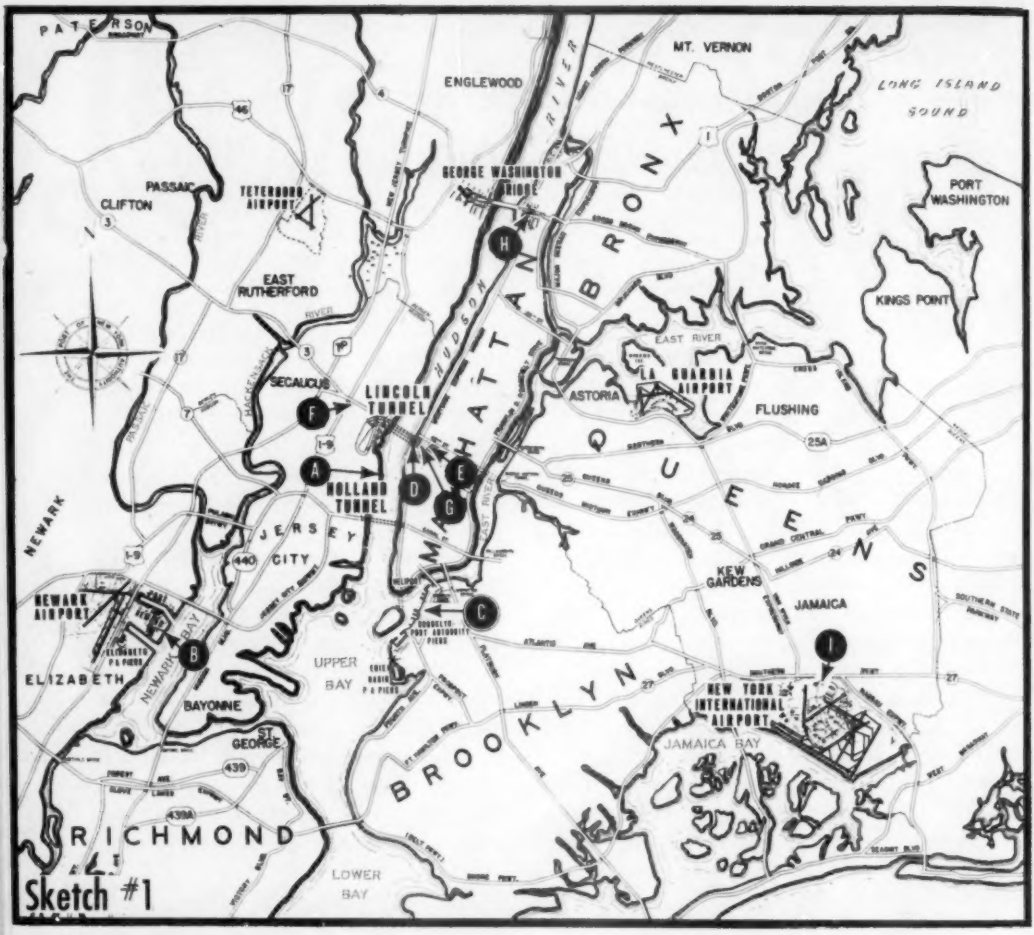


think this group might find interesting.

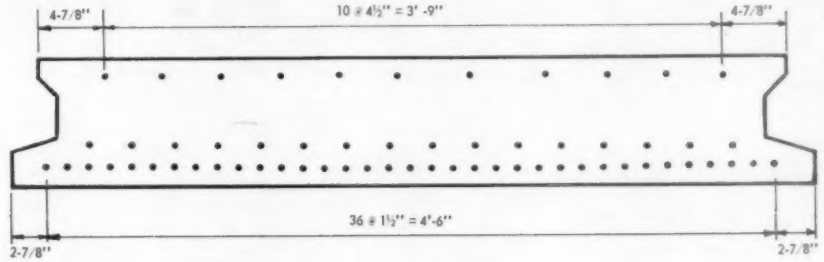
Approximately ten years ago, it was my privilege to spend a considerable amount of time with one of the distinguished innovators in this field. His name was Professor Gustav Magnel of the University of Ghent. He and Eugene Freyssinet were probably the pioneers in the development of precast concepts which produced such revolutionary changes in the design and construction of concrete structures in Europe. Like almost everyone exposed to the magic of Magnel's mind, I was so impressed with the giant strides he had made in the development of the theory and practice of prestressed concrete that I returned to this country fired with the confirmed conviction that, when at all feasible, new construction should be accomplished with the use of prestressed concrete. At about that time, The Port of New York Authority was embarking on a gigantic new program for the development of what was to become the world's largest airport—New York International. I had seen Magnel's one inch square thirty foot long column which vibrated like a harpstring but maintained its integrity, his "flying" bridges which spanned great widths with a minimum concrete member size, his industrial

buildings, and revolutionary new hangar designs at Melsbroek Airport.

However, our early efforts to introduce prestressed concrete into the construction at our various airports was not successful. Steel framed buildings were invariably lower in cost for structures bid competitively on prestressed concrete and steel designs. This is perhaps explained by the fact that contractors at that time were unfamiliar with economical prestressing methods and also that some of the techniques in vogue were complicated, time consuming and involved very high labor costs.

The first project of major magnitude, in which we were able to justify economically the use of prestressed concrete elements, was Pier "C" in Hoboken. This structure, resting on steel H-piles in the Hudson River, was designed for the Port Authority by Parsons, Brinckerhoff, Hall & MacDonald. The piles were capped with reinforced concrete bents. On these were laid 19 ft. 6 in. x 5 ft. x 12 in. thick precast, prestressed deck members (Sketch #2). Subsequently, the entire pier was knitted together by post-tensioned cables throughout its width. To validate the design and the execution of the precasting operation, a number of the concrete elements were subjected to destructive testing. Although in theory, the members should have cracked at a test load of ten tons, it was necessary to apply fifteen tons before the first crack appeared. A final rupture was not reached until a load of thirty tons had been applied.

Two more piers were designed and constructed using similar precast elements. One in Hoboken and one at the Foot of 38th Street in Manhattan. On subsequent piers built in Brooklyn, alternate designs were developed using reduced spans and substituting less expensive timber piles for the H-piles. In Brooklyn, several piers were bid competitively under the two systems and savings in deck construction of approximately 20 per cent were achieved. So, in the area of "finger" pier development, the apparent advantages in the use of prestressed elements have been more than offset



STRESS 10,000 LBS. PER STRAND
65 - 5/16" ø STRANDS

HOBOKEN
PORT AUTHORITY PIERS

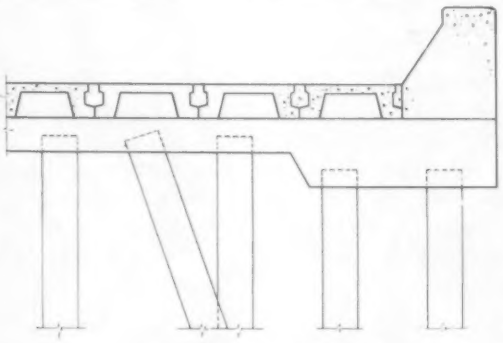
Sketch #2

by the economic penalties.

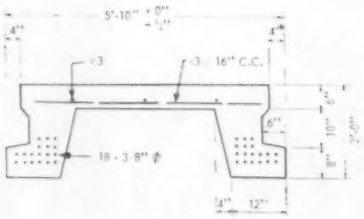
The next area of exploration was the relieving platforms along the channels in Port Newark and subsequently at the Elizabeth-Port Authority Piers. Here again, our early explorations indicated that substantial economies might be achieved by the use of a low level platform with precast concrete elements forming the deck of the substructure. Between 1949 and 1957, a number of these structures were erected using precast, prestressed elements. The basic design (Sketch #3) indicates the development of this construction.

We are now going forward with additional construction at Elizabeth and find that it is more economical to build such a structure using timber piles, poured-in-place concrete caps with either precast or poured-in-place deck slabs. Although the price differential between this construction and the design using longer spans and prestress elements is only two to three per cent, in such a highly competitive area as rental piers, the savings are significant.

Several unique problems which appeared to lend themselves to prestressed techniques evolved in the development of the Third Tube of the Lincoln Tunnel. Among them was the widening of an existing



RELIEVING PLATFORM



STRESS - 14,000 LBS. PER STRAND

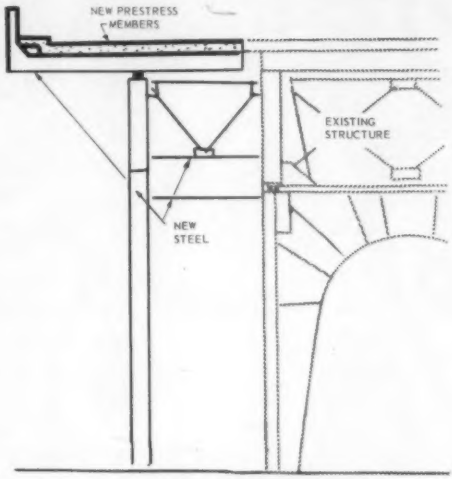
Sketch #3 PORT NEWARK

roadway over a railroad. Because of the heavy traffic on this facility, it was not possible to take the original roadway out of service while the structures were being modified. The existing road was a concrete deck resting on a series of portal braced steel bents. The original structure was quite whippy and, under traffic, the deck developed considerable oscillation. It was felt that this oscillation would interfere with the initial set of any poured-in-place concrete. Sketch #4 indicates the solution finally achieved. In the first instance, new steel was welded as an outrigger to the existing steel structure and precast, prestressed concrete units were placed on the steel and fastened down with tie bolts. (In the test of these concrete units we found that, although they were of minimum depth, they actually tested to 350 per cent of the maximum theoretical loading without failing.) After the units were in place, a two inch leveling course of asphaltic concrete was placed and rolled to secure a uniform wearing surface. Although, in this instance, the precast units cost considerably more than the conventional poured-in-place deck would have cost, it was possible to keep the original roadways open to traffic during widening operations.

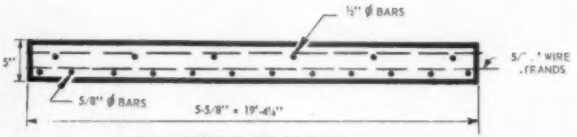
At the other end of the same project, it was necessary to erect a new steel frame bridge over the yards of the Pennsylvania Railroad between 31st and 33rd Streets in Manhattan. Using conventional construction, it would have been necessary to install concrete forms immediately adjacent to the railroad catenary system. Even with the greatest of care, there was a reasonable chance the workmen placing or stripping the forms might come in contact with the high voltage wires. Using the same technique that I have just outlined, precast, prestressed units were installed from the top of a steel deck structure, keeping all workmen well above the catenary and avoiding interference with the railroad operation. As before, when the units were in place, an asphaltic leveling course was installed to achieve good riding qualities in the road.

Approaches to tunnels in the New York Metropolitan Area historically had been constructed using concrete retaining walls. In some cases, they were huge gravity walls with "L" or inverted "T" footings. In others, particularly where there was a substantial amount of hydraulic uplift, a "U" section with a very heavy bottom slab was used. In the case of the new approaches to the Lincoln Tunnel, it appeared that prestressed concrete might be used to advantage in making the walls thinner and more durable. Sketch #5 shows how the horizontal prestressed wall elements were installed between counterforts. In this case, although economies were achieved in excavation, on account of the peculiar labor union conditions prevailing in the Metropolitan Area, it is doubtful if there was any substantial overall economy. It is probable, however, that the prestressed elements which were manufactured in a plant and cured under ideal conditions will prove to be more durable.

For the newest construction at the George Washington Bridge, we have incorporated a number of conventional prestressed elements into our design. The box girders which span Riverside Drive on the New York approaches to the Bridge are shown in Sketch #6. We are using deep I-beam shaped girders for our north and south Avenue bridges where the new expressway across Manhattan is depressed under the existing



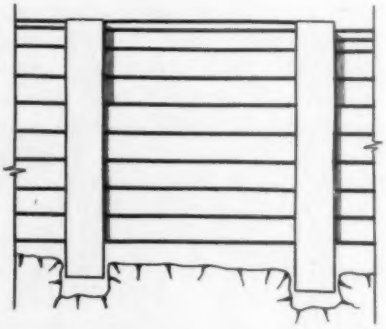
DECK STRUCTURE



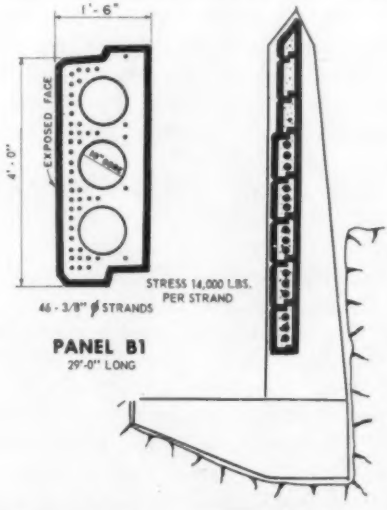
STRESS - 9,000 LBS. PER 5/16" STRAND

LINCOLN TUNNEL
APPROACH
(OVER SUSQUEHANNA R. R.)

Sketch #4



ELEVATION



PANEL B1
29'-0" LONG

SECTION

LINCOLN TUNNEL
WALL STRUCTURE

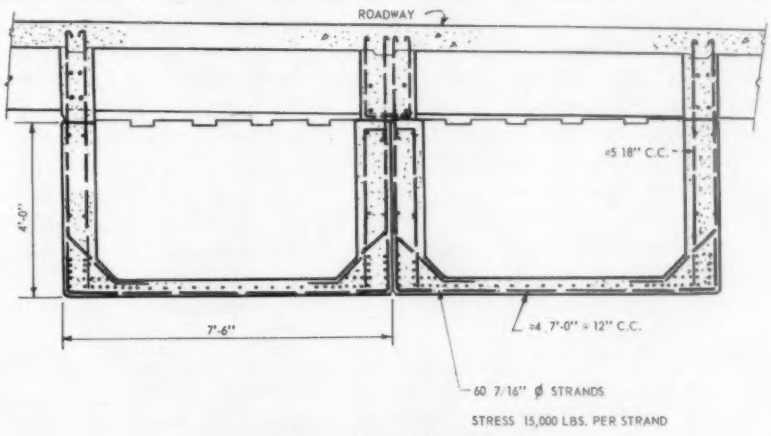
Sketch #5

Avenues. Contractors working on the project are permitted to elect the method used in stressing the girders and the deck slabs. Although the techniques adopted by the two main contractors on the project differ, both will achieve the same end result. The design, which makes provision for not only supporting the bridge but carrying public utilities across the cut, is shown on Sketch #7. At the time of the original bidding on the project, steel was in short supply and the prestressed alternates appeared to offer certain advantages. On future projects, it will be necessary to do a careful analysis to see whether the substantial savings in maintenance, achieved by using concrete in lieu of structural steel, will justify the higher initial cost. In this connection, it might be interesting to note that steel designers, taking a leaf from the prestressed book, are actually achieving economies in structural steel design by the use of prestressed cables or rods, particularly on long span structures where height considerations are important.

Concerning airport construction, our record of achievements with prestressed concrete is not spectacular. Although the Port Authority has designed and bid a number of prestressed concrete hangars, in every case, the design employing conventional structural steel has proved to be more economical.

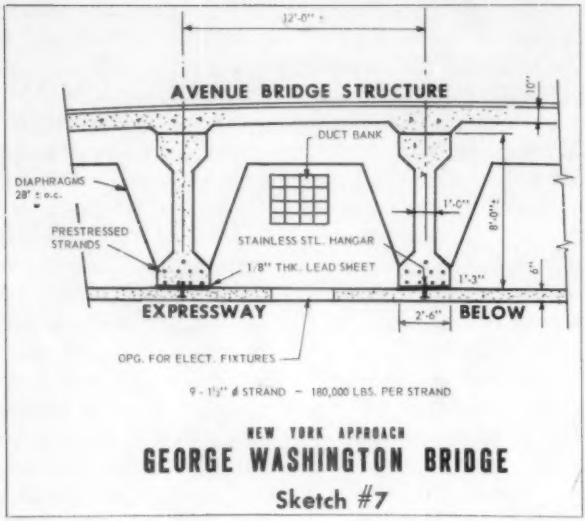
We have, however, installed a number of prestressed concrete bridges in the road circulation system. These were erected at the time of the acute steel shortages and were the result of a substitution requested by the contractor. He reports that their use was dictated by the tight time schedule of the project, not because there were any substantial economies resulting from the use of prestressed members.

One other area, for which I had high hopes, was the use of prestressed concrete for runway paving. Although there have been a great many examples of the use of this material, we can find no case in which any real economies were achieved on reasonably stable subgrades. We are fortunate at New York International Airport in that we have an eight foot layer of



NEW YORK APPROACH

Sketch #6 **GEORGE WASHINGTON BRIDGE**



hydraulically placed sand fill over the entire area. At least our bearing conditions are uniformly good. Although we have developed some designs and budget estimates for prestressed pavements, the costs have run substantially more than the conventional concrete pavements presently being used. I think it is safe to predict that, in the foreseeable future, a technique can be developed which will make prestressed concrete for runway pavements competitive with the conventional pavements now being installed. To date, nothing of this nature has been publicly reported or exhibited.

To summarize then, prestressed techniques have been used on every facet of the Port Authority's construction program. I think it is significant that where prestressing has been used to replace conventional construction the development of newer techniques and design theories applied to conventional construction materials have resulted in lower costs. However, where specialized problems are encountered, prestress has very often proved to be the only valid solution. It is therefore my opinion that, if the prestressed industry is to go forward, it is necessary to develop new and different uses for the product. It must improve the technology of the art both from a design and from a construction standpoint, and, of course, to make every effort to do away with any artificial barriers which result from outmoded practices or regulations. In brief, our infant industry, although it has achieved a third of a century of experience, still has growing pains.

This then is the challenge to the industry. How to find new and different uses for prestressed concrete when conventional construction is proving either unsatisfactory or uneconomical. In the latter category, it is important, if this industry is to succeed, for our designers to develop new and better methods of precasting, prestressing, and erection; and for the contractors and labor unions involved, to develop a modus operandi under which this new and unique product can be exploited to its full potential. If the industry is to survive and prosper, these obstacles must be met and overcome.

Manhattan's Prestressed Piers

Presented at the Prestressed Concrete Institute Convention
September, 1960

by **Charles Prokop***

Economical building construction with prestressed concrete involving large quantities of long span units must start with a thorough planning by the engineer.

Planning by the engineer is perhaps the most important phase which determines the cost of any structure. Planning must have simplicity and uniformity if a structure is to be built economically, which can only be achieved by using a little more concrete and steel than required by design.

Most engineers design members to use the least amount of concrete and steel and believe this will develop the most economical structure. This is not always so.

During the late 1930's and early 1940's, we constructed many reinforced concrete structural frames for low rental housing projects. These structural frames were made up of beams and slabs. The beams were arranged so that they were always around the perimeter of the rooms.

During this period, engineers used this system believing this was the most economical structure since it used the least amount of materials.

Along in the late 1940's, long before flat plate design, one engineer with some imagination designed a thickened concrete slab with a beam within this slab, thereby eliminating all beams except for spandral beams and beams around openings. In order for this engineer to design such a structure, obviously he had to use more materials.

In making cost comparisons of the two types of construction built around the same time, it was discovered that the structure with more materials had the lowest cost upon the completion of the job.

I believe this same theory can apply to pre-cast prestressed concrete; that is, use more materials in order to obtain simplification and uniformity so as to obtain lower construction costs. To achieve this, particularly in girder and beam construction, I believe it can be accomplished by a few simple rules:

First of all, you must establish an efficient depth of girder and hold it constant. Use the same shape for all the girders. This also applies to sections of the beam as well. Another point would be to keep the elevation between the tops of the beams and the tops of the girders at a constant distance; not necessarily at the same level, but at a constant distance. This will keep all of the beam copes the same shape, and it will keep brackets on the girders the same height.

Another point is that the beam spacing should be held uniform. The girder centers should be held uniform, including those adjacent to expansion joints. If that is done, these beams will all be the same length.

Now, there is also one other prob-

*Vice President
Corbetta Construction Co.
New York City

lem that we have in the field and that is variations, particularly in bearings. Adequate bearing areas must be provided for several reasons: At times of prestressing beams foreshorten; they lose a certain amount of bearing at these points. Sometimes beams and girders have holes cast in them and they lose bearing at these points. I think that the engineers in their planning should allow for such variations. This seems to be a constant problem in all prestressed concrete work.

Of course, I don't really believe you can obtain a hundred per cent efficiency, as I have mentioned here, but I do believe you can obtain, say, 75 per cent of effective efficiency by good planning, and I am sure that it will show a saving in construction costs.

I will now discuss the construction of Pier 40, built for the Holland-American Line. The owners are the Department of M & A; Roberts and Schaefer are the architects and engineers. The lowest concrete bid was $3,977,000; the lowest steel fireproof alternate was $4,250,000. The Corbetta Construction Company was low on both bids.

This job is divided into five types of construction, interwoven into one massive building. There are:

Precast channel slabs.
Precast pre-tensioned beams.
Precast post-tension girders.
Precast Girders.
Poured in place concrete construction.

There is a total of 27,300 cu. yards of concrete in this structure, of which one-half is in the precast and prestress items.

There is a total of 10,283 precast channel slabs in this structure. Figure 2 is a photo of the precast channel slab yard at one end of the site. Here you see 119 molds in four lines, of 57 different sizes and shapes of molds, with adjustable end bulkheads on some molds. These molds produced 77 different slabs. The 119 molds were filled with 3,000 psi lightweight concrete, using sand, and high early strength cement. When the concrete strength reached 1500 pounds per square inch, the slabs were removed from the molds. The molds were cleaned and oiled. Reinforcing steel and prebent wire mesh installed and concrete placed, all in a single day.

The truck court area of the pier was capable of storing about 5,000

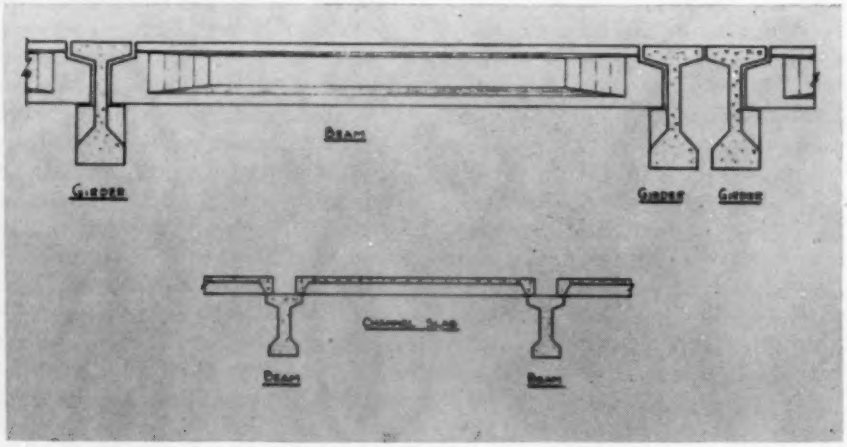


Fig. 1—Structure framing details

Fig. 2—Slab casting yard

slabs at one time. Each slab, as removed from the molds, had to be classified and moved to the storing area to a predetermined stack. At the time of erection, 100 slabs of about 30 different types, were picked out in sequence of erection. You can readily see the amount of record keeping and supervision that was required to keep track of ten thousand slabs of 77 different types. (See Fig. 3)

Figures 4, 5, and 6 show one of the methods that was used to erect the precast channel slabs.

Figure 7 shows the way the ceiling looks with the channel slabs, prestress beams, and the girders in place. It required 66 shop line drawings to express this work to the men in the field. The Precrete Company, manufacturers of precast prestressed units, furnished and delivered all pre-tensioned beams for this job.

There are approximately 1800 prestressed beams in three sizes—30 inches, 36 inches, and 42 inches in depth; predominantly 30-inch beams. There were 233 special beams; all beams were 42 feet long. There were 263 types of variation in the beams; primarily lengths, and cope outs in top and bottom. Each beam was marked and set in accordance with an erection drawing.

The beams arrived on the job by truck in sequence of erection, and I might indicate here that the manufacturer did an excellent job, which eliminated rehandling and sorting of beams on site.

In order to express the members in figure 8 to the men in the field, it required 100 shop drawings. There are 353 post-tensioned prestressed girders; 24 double cantilevers, 95 single cantilevers, and 234 simple spans. The simple spans generally are about 50 foot span. It required 38 sets of wood forms to produce the 353 girders at a rate of three per day every day. The forms were used nine to ten times with a great deal of modification necessary in order to reuse them.

Figure 9 shows the last girders on the job, so you can count up and see all the variations that have to be taken as they moved through the project. Each form, when stripped and moved to the next location, invariably had to be modified in one form or another. Changing some locations of beam brackets, bracket heights, omission of brackets, length girders, bulkheads, and so on.

You cannot imagine the amount of work required to express each change to the man in the field; the time it consumed, supervision, double checking, to be sure the work was done correctly, and even then we missed some details. In order to

Fig. 3

VOLUME TIGHTLY BOUND

—Slab erection sequence

Fig. 4

Fig. 5

Fig. 6

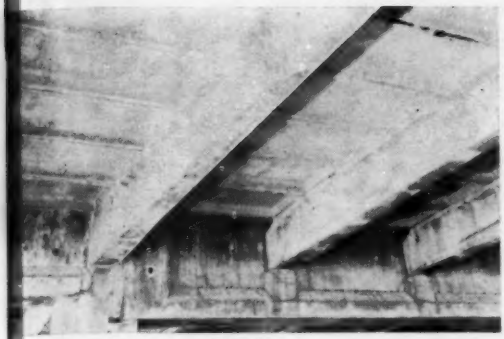


Fig. 7

express this information to the field personnel, it required 600 shop drawings. The Prescon Company furnished all the tendons for the girders. They made 13 shop drawings for the post tensioned tendons alone.

Figure 9 shows the precast girders as they were delivered to their predetermined layout on the deck. They also had to be arranged so that they would not overload the deck and so that they would be accessible to truck cranes.

Fig. 8

Figure 10 illustrates a double cantilever beam in the background.

Figure 11 is a very good picture. It shows simplicity, it shows uniformity, and it is simple. Compare it with

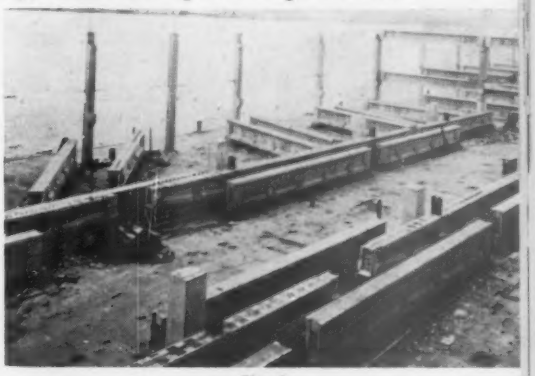


Fig. 9

Figure 12, another part of the structure, with its many complex details and connections.

Figure 13 is view just short of the end of the pier, for a distance of some 600 feet.

The spans pictured here are 50 feet. The cantilever on one side is 25 feet. Quite impressive, isn't it? And it is all prestressed concrete, excepting for the wooden-

Fig. 10

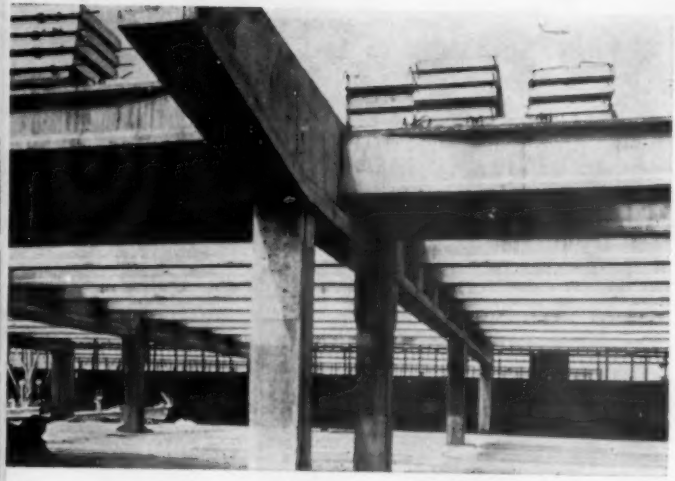


Fig. 11

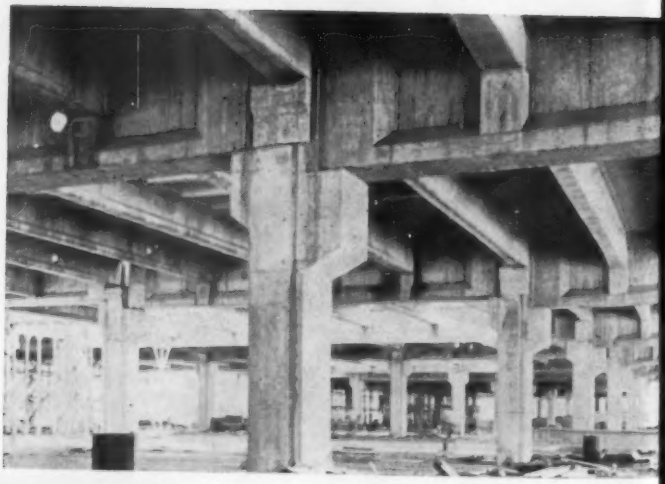


Fig. 12

Fig. 13

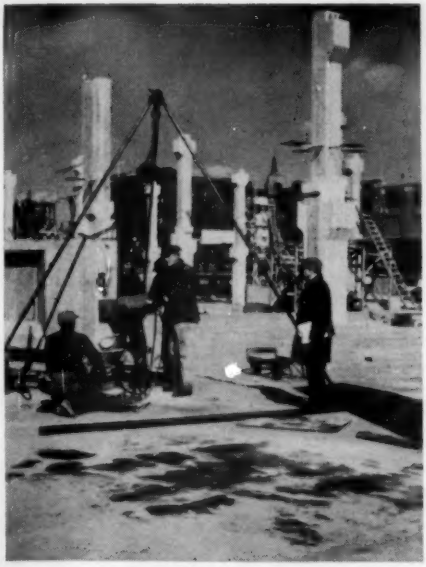


Fig. 14

faced slab.

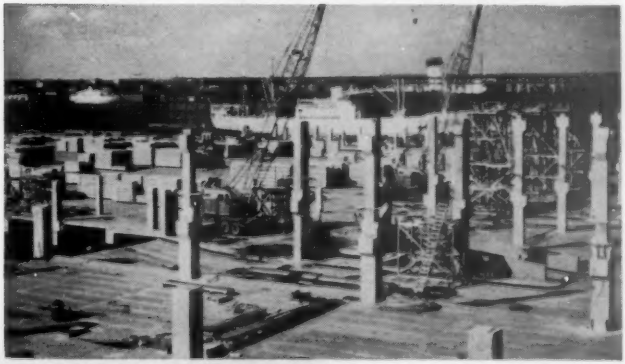
Figure 14 shows a jacking unit crew, post-tensioning a girder. I'd like to focus your attention to the detail of the column immediately behind the man on the right. Isn't that impressive?

Because of the many variations, the accurate placing of the columns presented some difficulty also. Careful field supervision was necessary. (Fig. 15) You can imagine the work that had to be done in order to get these in the correct location.

Figure 16 shows the laying of the slab reinforcement placed in order to unite the entire structure.

In conclusion, I would like to say this: In order for the prestress industry to move forward, we must first look back and utilize all of our past experiences and maybe change some of our thinking. We should possibly strive for simplification and unification by using more concrete and steel in order to obtain better construction costs.

Fig. 15



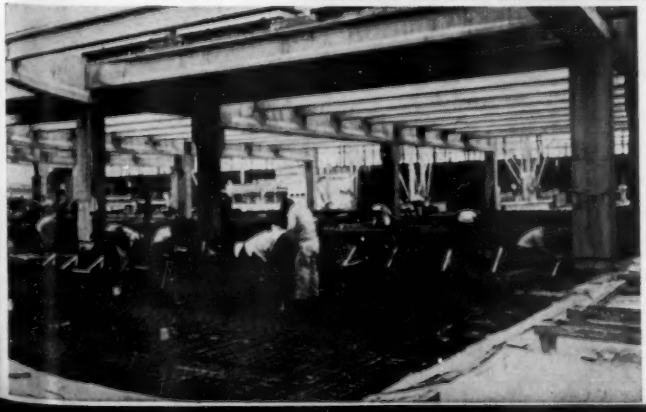


Fig. 16

Positioning Drape Points In Prestressed Members

by DAVID MORRIS*

The method described herein offers an expedient means of locating a predetermined force along the longitudinal plan profile of a prestressed concrete member. The study is particularly applicable to a precast pretensioned member in which "draping", "hold-up", or "hold down" devices are employed to change the direction or spacing of the strands. These mechanical means are often necessary in order to vary the eccentricity of the prestressing force and thus meet certain design criteria, i.e., to lessen tensile stresses at the end of a member, to reduce camber, or to increase the internal resisting moment. When draping the strands is required, it shall be shown that it is sometimes desirable to employ a double drape scheme such as shown in Figure 1a.

Basically, the profile position of the prestressing force is governed by the shape of the bending moment curve of the external loads. This external moment diagram must be enveloped by the internal moment diagram at every point along the length of the member in order to prevent excessive fiber stresses from

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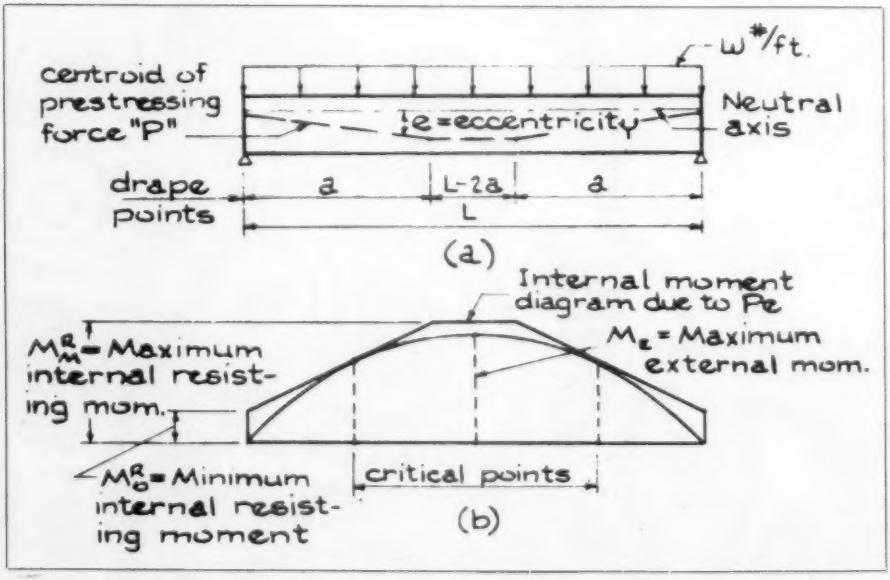

Fig. 1

occurring. As shown in Figure 1b, the critical point may not be the position of maximum moment. Thus, if the eccentricity of the prestressing force varies lineally between the points of maximum and minimum external moment, the intermediate points along the member should be checked to ensure that the allowable stresses are not exceeded.

A widely used method of accomplishing this task is the graphical approach in which the two moment diagrams are drawn to a scale and superimposed on the same sketch. Here the points of maximum and minimum resisting moment are connected in a manner that encloses the external moment curve. This may indicate that a straight line variation between the points is sufficient or, as shown in Figure 1b, the resulting diagram may call for an extension of the region of maximum resisting moment. This is accomplished by the double drape system. If this is required, then it is frequently advantageous, from a standpoint of prestress losses and camber considerations, to keep the drape points close together and as far away as possible from the ends of the member. This results in the sloping internal moment line being drawn tangent to the external moment curve and intersecting the maximum internal moment line at some point. The approximate location of this point can be found by scaling its distance from the end of the beam. While the graphical method is usually satisfactorily accurate, it does require time consuming plotting of a parabolic curve.

The following study provides a theoretically correct equation which determines, first, whether one or two drape points are required, and second, locates the required points with respect to the end of the member. A chart showing the equation in graphical form is also provided. The study is based upon minimum design requirements and is applicable to cases in which:

1. Members are of uniform cross section.
2. Members are uniformly loaded.
3. Members require one or two strand drape points.
4. Uniform prestress forces are assumed to exist all along the member.

In order to reduce the study to a unitless form which will be applicable to any case, the minimum resisting internal moment, M^R_o, shall be divided by the maximum dead-live load uniform external moment, M_E, so that

$$K_o = \frac{M^R_o}{M_E}.$$

Similarly, the maximum resisting internal moment at the point of M_E shall be called M^R_M and,

$$K_M = \frac{M^R_M}{M_E}.$$

For M_E to be unitless and still keep its proporting to M^R_o and M^R_M, its maximum ordinate shall now be the base "1", or

$$\frac{M_E}{M_E} = 1.$$

It can be seen from Figure 2 that if K_{o_1}, and K_{M_1}, are of sufficient magnitude, a line connecting them, Line #1, will fall above the second degree parabolic curve representing the unitless bending moment curve. This, in effect, means that the resisting moments at the points of zero and maximum dead-live load moment are great enough to insure complete envelopment of the curve and consequently a single drape

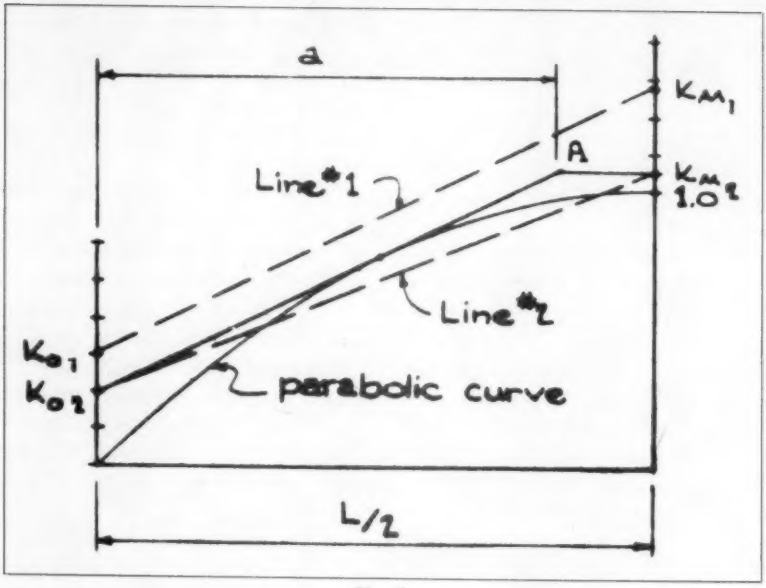

Fig. 2

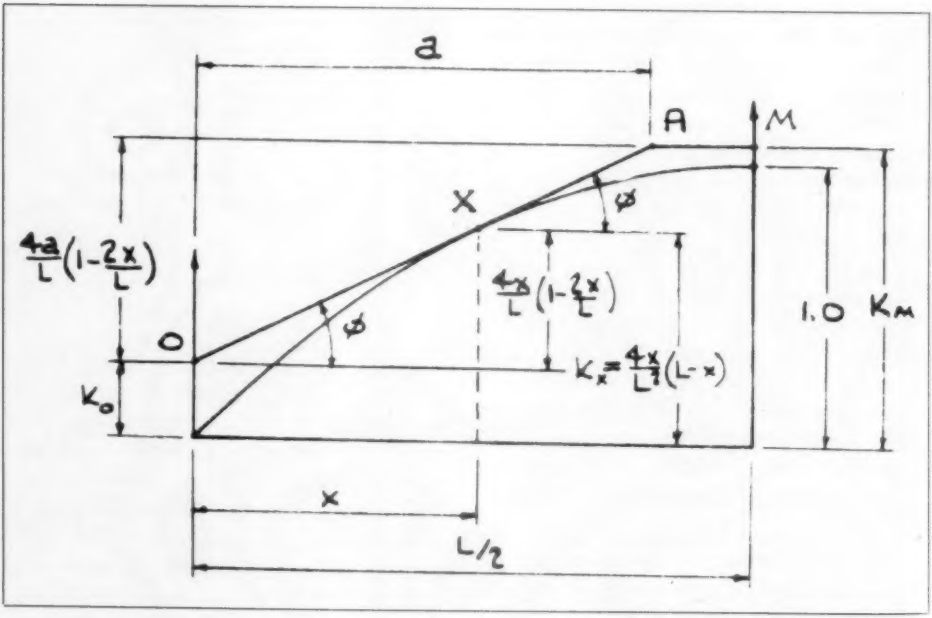

Fig. 3

point is sufficient. Suppose, on the other hand, that resisting moments are applied which result in the values of K_{o_2} and K_{M_2}, and are such that their moment envelope, Line #2, does not satisfy the requirements. It then becomes necessary to either increase one or both K values (by increasing their respective M^R's) or to add an additional drape point. The latter solution produces a resisting moment diagram which encloses the parabolic curve within three lines; two tangent to the curve from points K_{o_2}, the other horizontal from K_{M_2} and extended to intersect the first lines at point "A". These points represent the absolute maximum distance, "a", that a pair of hold downs or drape points may be placed with respect to the points of zero moment. If an equation can be written in terms of "a", K_O and K_M, then for any given values of M^R_O, M^R_M, and M_E, a quick evaluation can be made to determine the number and position of the required drape points.

Referring to Figure 3, the equation for the uniform load curve shall be expressed in unitless form as,

$$K_x = M_x/M_E = \frac{W_x}{2M_E}(L\text{-}x) \quad \ldots\ldots\ldots (1).$$

If the maximum ordinate or the point of maximum moment is given by

$$M_E = \frac{WL^2}{8}$$

Therefore:

$$W = \frac{8M_E}{L^2}$$

and substituting in Equation (1),

$$M_x/M_E = \frac{8M_E}{L^2}\frac{X}{2M_E}[1\text{-}x] = K_x$$

$$K_x = \frac{4x}{L^2}[L\text{-}x] \quad \ldots\ldots\ldots\ldots\ldots\ldots (2).$$

If the line OXA is drawn tangent to the curve at point X, then the slope at this point is

$$\frac{d(K_x)}{dx} = \frac{d}{dx}\left(\frac{4x}{L^2}[L\text{-}x]\right) = \Phi$$

or

$$\Phi = \frac{4L}{L^2} - \frac{8x}{L^2} = \frac{4}{L}\left[1 - \frac{2x}{L}\right] \quad \ldots\ldots\ldots (3)$$

Since the line OXA originates from point "O" (defined by the value K_O) and is tangent to the curve at "X", a second equation for K_X can be written as the sum of,

$$K_x = K_o + \Phi\,(x)$$

$$K_x = K_o + \frac{4x}{L}\left(1 - \frac{2x}{L}\right) \quad \ldots\ldots\ldots\ldots (4)$$

Combining Equations (2) and (4),

$$K_o + \frac{4x}{L}\left(1 - \frac{2x}{L}\right) = \frac{4x}{L^2}(L\text{-}x),$$

and rearranging terms,

$$K_o = \frac{4x}{L} - \frac{4x^2}{L^2} - \frac{4x}{L} + \frac{8x^2}{L^2}$$

$$K_o = 4x^2/L^2$$

Therefore,

$$x^2 = \frac{L^2}{4}K_o,$$

or

$$x = \frac{L}{2}\sqrt{K_o} \quad \ldots\ldots\ldots\ldots\ldots (5)$$

Observing that the point in question, "A", Figure 3, is defined by the intersection of lines OXA and AM, it can be seen that this point lies along an ordinate equal to K_M and on an abscissa at distance "a" away from the point of zero external moment.

By defining the ordinate, K_M, as the sum of,

$$K_o + a\Phi = K_M,$$

substituting Equation (3) for Φ

$$K_M = K_o + a\left[\frac{4}{L}\left(1 - 2\frac{x}{L}\right)\right]$$

and substituting Equation (5) for x,

$$K_M = K_o + \frac{4a}{L}\left(1 - \frac{2}{L} \cdot \frac{L}{2}\sqrt{K_o}\right),$$

$$K_M = K_o + \frac{4a}{L}(1 - \sqrt{K_o}) \quad \ldots\ldots (6).$$

This may be rewritten so that

$$a = \frac{K_M - K_o}{4(1 - \sqrt{K_o})}(L) \quad \ldots\ldots\ldots (7),$$

or substituting in terms of actual moments,

$$a = \frac{M_M^R - M_o^R}{4M_E\left(1 - \sqrt{\frac{M_o^R}{M_E}}\right)}(L) \quad \ldots\ldots (8).$$

Either of the two preceding equations may be used to determine "a". It can be seen from Equation (7) that by substituting values for any two of the three unknowns, K_o, K_M or "a", the third may be found. If limits are established for these unknowns then they may be plotted graphically.

It is apparent that the distance "a" can have maximum relative value of .50L, or in other words, a single drape point is all that is necessary when "a" is equal to .50L or more. The minimum value of "a" could approach zero, however, a close examination of Figure 3 will show that when "x" is equal to zero, "a" is as small as it will ever need be in order to satisfy the design requirements. In this case, using x = 0 in Equation (5),

$$x = \frac{L}{2}\sqrt{K_o} = 0, \quad K_o = 0.$$

Substituting in Equation (7),

$$a = \frac{K_M - 0}{4(1 - 0)}(L) = K_M\frac{L}{4} \quad \ldots\ldots (9),$$

and, from inspection, "a" is minimum when K_M is minimum and if K_M is 1.0,

$$a\ (\text{min.}) = L/4 = .25L.$$

The minimum value of K_o is zero since M_o^R may also be zero. An actual maximum value of M_o^R could be as high as M_E and in this case K_o would be equal to K_M. Further inspection of Figure 3 shows, however, that if M^R is as high as M_E then the resisting envelope becomes tangent to the curve at the point

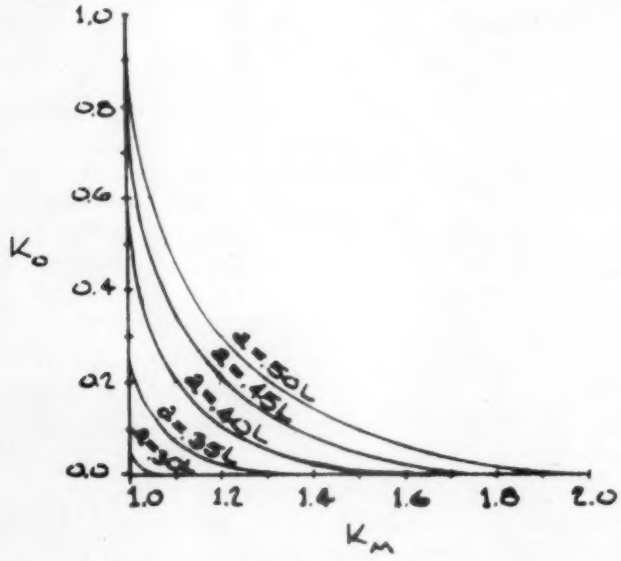

Fig. 4

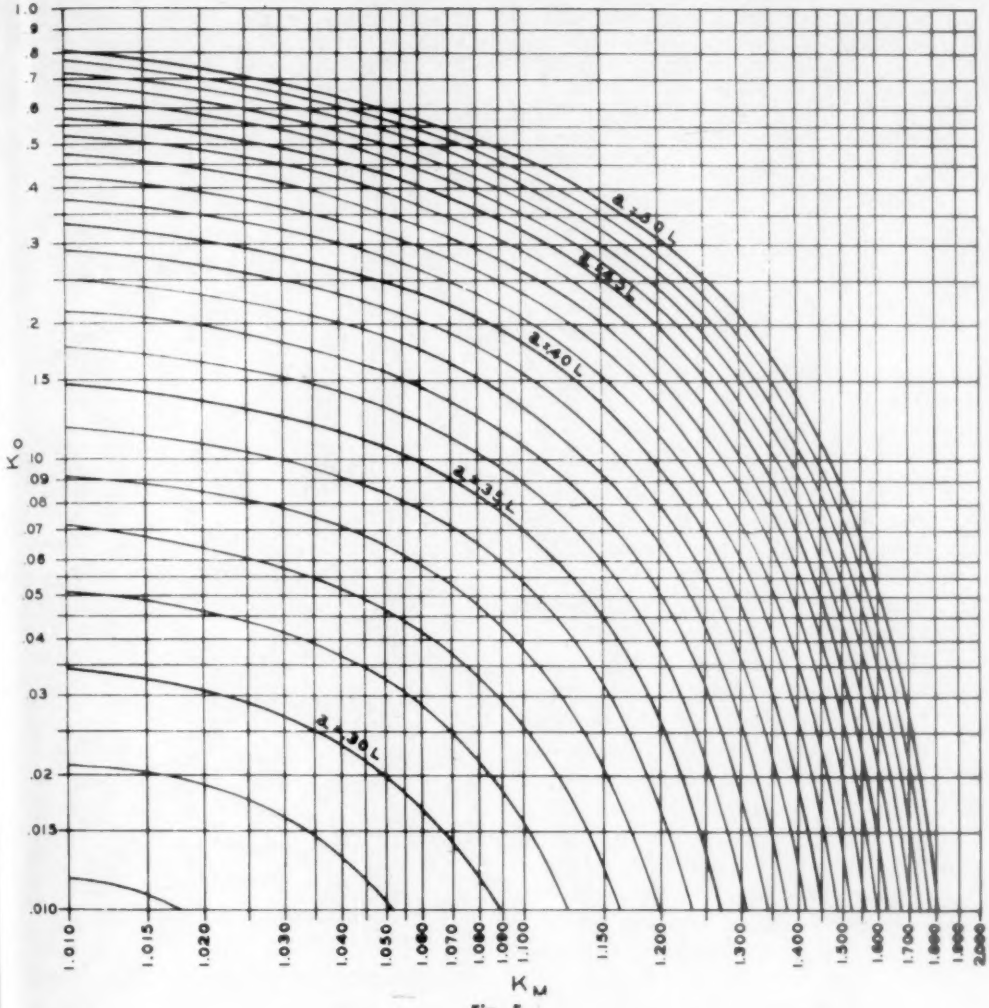

Fig. 5

of maximum external moment, Φ is zero, and the design requirements are fulfilled. Thus for practical purposes,

$$K_O \text{ (max)} = 1.0.$$

The minimum value of K_M is one and exists when $M^R_M = M_E$. In order to establish a required maximum (though not a possible maximum), Figure 3 shall be used again. Observe that K_M is at a maximum when the point of tangency is at $x = 0$ (Φ max.), and $a = .50L$. Since $K_O = 0$ when $x = 0$, Equation (9) now applies and can be written as

$$K_M = a\frac{4}{L} \quad ,$$

and $$K_M \text{ (max.)} = a \text{ (max)} \frac{4}{L} = \frac{L}{2}\frac{4}{L}$$

$$K_M \text{ (max.)} = 2.0.$$

Summarizing the practical limits:

$$.25L \leqq a \leqq .50L,$$
$$0 \leqq K_O \leqq 1.0,$$
$$1.0 \leqq K_M \leqq 2.0.$$

Figure 4 is a graphical representation of equation (7) in which the above limits are imposed upon the unknowns.

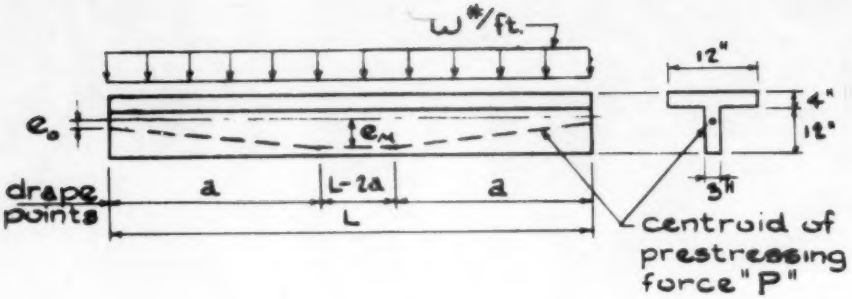

Fig. 6

Here the "a" curves are so closely grouped that a close interpretation is difficult to make. Therefore the same curves, shown plotted to the two cycle log scale in Figure 5, are thought to better illustrate the effects of the changes in "a" with various values of K_O and K_M. Though this chart offers much greater accuracy than the one shown in Figure 4, it has one disadvantage. That is, being logarithmic in nature, zero quantities cannot be plotted. However, as shown in the chart, the two K quantities are within 1% of their minimum limits and it is thought that most practical problems will fall within the range of those shown. Any set of K values not readable within the chart limits, and not falling obviously outside the a = .50L limit, may be quickly checked by the basic equation,

$$a = \frac{K_M - K_O}{4(1 - \sqrt{K_O})}(L).$$

For example, the tee beam shown in Figure 6 shall be checked to determine if an assumed single drape point at midspan is sufficient.

Let,

$W = 300$ lb/ft
$L = 40$ ft
$M_E = W L^2/8$
$= 60$ ft kips $= 720$ in kips
$P = 67{,}200$ lb (after losses')
$e_o = 1.50$ in
$e_M = 8.25$ in
$y_b = 10.57$ in
$y_t = 5.43$ in
$A = 84$ in^2
$I = 1813$ in^4
$S_t = 334.0$ in^3
$S_b = 171.5$ in^3
$f'_c = 5000$ psi

The maximum fiber stresses due to the external moment M_E are:

$f_{top} = M_E/S_t = 720/334 = 2155$ psi (comp.).
$f_{bot} = M_E/S_b = 720/171.5 = 4198$ psi (ten.).

The stresses resulting from the prestress force "P", acting eccentrically through distance e_M, at midspan are:

$f_{top} = P/A - Pe_M/S_t$
$= 67200/84 - 67200(8.250/334$
$= -860$ psi,
$f_{bot} = P/A + Pe_M/S_t$
$= 67200/84 + 67200(8.25)/171.5$
$= 4033$ psi.

Assuming the allowable tension is governed by

$6\sqrt{f'_c} = 6\sqrt{5000} = -423$ psi,

then

$-4198 + 4033 = -165 < -423$psi allowable,

and the design is satisfactory.

The maximum resisting moment at the drape point or points is thus

$M^R_M = f_b S_b = (423 + 4033)(171.5)$
$= 764{,}200$ in.lb.

The factor K_M is then found as

$K_M = M^R_M / M_E = 764.2/720 = 1.0613.$

Similarly, the stresses due to P acting at e_o at the ends of the members are:

$f_{top} = P/A - Pe_o/S_t$
$= 67200/84 - 67200(1.50)/334 = 499$psi,

$f_{bot} = P/A + Pe_o/S_b$
$= 67200/84 + 67200(1.50)/171.5 =$
1388psi.

$M_o^R = (423 + 1388)(171.5) = 310{,}587,$

$K_o = 310.587/720 = .4313.$

And then,

$$a = \frac{K_M - K_o}{4(1 - \sqrt{K_o})}(L) = \frac{1.063 - 0.4313}{4(1 - \sqrt{.4313})}$$
$$= .4587L$$

Since,

$.458 < .50L,$

two drape points are required and are located at a = .4587(40) = 18.35 feet from the ends of the member.

The chart in Figure 5 also gives the answer, a = .459L (last place interpolated).

The preceding example makes use of a simply supported beam in which the maximum and minimum moments occur at midspan and at the ends respectively. It should be pointed out that the method shown is not limited to simple spans and can be applied to any structural system once the points of maximum and minimum positive moments due to uniform loads are known. The method does not, of course, cover the case of maximum negative moment and its prestress design. Here however, the designer can usually make a rapid and positive visual check.

Prestressed Concrete For Railroads- Past Experience And Future Possibilities

by F. R. Woolford*

Presented at the Prestressed Concrete Institute Convention
New York City, September, 1960

Acceptance and adoption of prestressed concrete by American railroad industries could provide a great impetus to the advancement of this construction material. Two prerequisites though, are necessary to make this prediction become a fact, and these are safety and economy. Throughout history, progress in all fields of human endeavor has been the result of factors of human need and economics. It follows, that progress in industrial enterprises would also be proportioned to the needs and economics of industry. This has been the case of the American railroads and their relationship to the modern design and use of concrete.

Concrete, as a construction material, predates the Christian era; and cements have steadily improved throughout the ages. Likewise, the idea of imposing stresses opposite to those which would be experienced under live load conditions is not confined to this century, although materials, other than concrete, were used in the basic processes. It would be presumptuous of me to elaborate on the long history of concrete or reinforced concrete, but I would like to say, as historical background and, as a plug for the home town, that the first application of prestressing to concrete, was in the year 1886 in San Francisco, California, by Mr. P. H. Jackson. The method was not satisfactory at that time because the steel utilized had insufficient strength and plastic properties. The modern concept of prestressing was not reached until high-strength steels became available and Monsieur Freyssinet, in 1926, established and stated the practical conditions under which preliminary stresses could be created in concrete members. At that time the railroads of the world began to seriously examine the process.

Speaking strictly in terms of the American railroads, the need for any design improvements was not apparent, even in the first quarter of the twentieth century. Construction materials, such as steel, concrete, timber, and clay products were available in great quantities and at economically sound prices. The abundance of raw materials made the manufacture of such products possible at economical costs. In the past, low material costs made an extra sack of cement or two pounds of steel a negligible item. Since the depression years these costs have increased, and so have manufacturing costs, to nearly double. Now it behooves all engineers, not only those of the railroads, but others as well, to sharpen their pencils and to strive for a method of modern structural design, which will insure both safe and economical structures and which

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will be suitable for their particular functions. This, of course, reflects the method proposed by Monsieur Freyssinet in his prestressed concrete.

The application of "prestress" principles, to railroad structures, culminates a history that parallels the growth of this country. The rise of our rail transportation network, from the early nineteenth century to the present time, has been a barometer of the industrial and political maturity of these United States. As expanding railroads fingered their way into newly-forged frontiers, the construction industries, and men who were charged with the responsibility of pushing the rails through the wilderness, developed and perfected many of the latest techniques in basic construction concepts of their days. Concrete, rubble masonry, timber, and steel were commonplace in bridge structures, tunnels and buildings. To the credit of these early railroad engineers, a great number of these structures are still in use on the main lines of America's rail transportation systems.

Traditionally, railroads have been extremely careful in applying any new design procedure for bridge structures that is radically different from the tried and proven accepted practice. This policy might appear reactionary to engineers who are not familiar with the design criteria for railroad structures. But if one considers the ratio of the live load to the dead load as applicable to a railroad bridge versus a similar ratio applied to a highway or other comparable loaded bridges, you will find the numerator and the denominator, in each case, nearly reversed. I am sure the members of this group can appreciate the importance of such a condition. Furthermore, as a public carrier, which must finance its own structures and also accept the responsibility for the safety of the public in railroad conveyances over such structures, consequently an attitude of conservatism, with regard to their design, must be followed. This does not mean that officers and engineers of railroads are opposed to progressive ideas. On the contrary, no one knows better than they the need for creative thinking and action, since only that will keep the industry a healthy and vital organ of this country's anatomy. It does mean though, that we cannot assume radical changes in policy without first applying sound engineering research techniques to such changes. This of necessity has led to the creation of a research group sponsored by all member railroads of the Association of American Railroads. You are no doubt aware of this group since its construction and maintenance section has contributed, over the past years, invaluable research data, not only for the railroad industry, but for the engineering and construction profession as a whole. In the late 1940's work was started in the laboratory of the American Railroad Engineering Association in Chicago, located adjacent to the campus of the Illinois Institute of Technology. This program was carried out under the direction of Mr. G. M. Magee, Director of Engineering Research. As a result of this preliminary study, prestressed concrete slabs were actually constructed and tested at the Bureau of Reclamation laboratory in Denver, Colorado. One slab, along with others containing conventional reinforcement, was laboratory tested to failure. Two additional slabs, of prestress design, were constructed simultaneously with the aforementioned slab, and after the laboratory slab proved satisfactory, they were placed in an

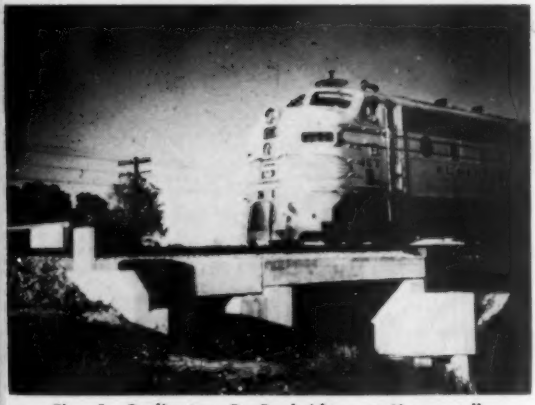


Fig. 1: Burlington R. R. bridge at Hunnewell, Mo. first use of prestressed concrete by railroads.

actual bridge for field testing under operating conditions in March of 1954 on the Burlington Railroad at Hunnewell, Missouri. (see fig. 1) Here too, field tests again demonstrated the value of the pretensioned structural members.

One of the assignments of A.R.E.A. Committee No. 30, working with Mr. Magee's group in 1953, was the investigation of impact and bridge stresses on various types of concrete beams. This work was done at the Lehigh University at Bethlehem, Pennsylvania. Six of these beams were pretensioned and subjected to normal static tests as well as fatigue tests. The results, in all respects checked reasonably well with critical fatigue loads as predicted. Recently two seventy-foot prestressed concrete girder spans were analyzed by this committee in conjunction with the A.R.E.A.'s masonry Committee No. 8. This structure was built by the Santa Fe Railroad at Colorado Springs and was designed to carry a Cooper's E-72 loading with a cast-in-place concrete ballast deck. (see fig. 2) These girders utilized the post-tensioning method, which, in some railroad engineers' opinion, is the only practical one for members of this size and shape.

At the same time as the Santa Fe test, an additional field test was progressed on a double track precast prestressed structure on the Southern Railway. The spans on the Southern were about 26 feet long, and consisted of 12 rectangular pretensioned beams. In all cases, results from these tests were so encouraging that prestressed concrete has become one of the most promising construction materials in our railroad industry. Research in this field has continued at an ever-increasing pace. I might add here, that an increasing interest has developed in the use of epoxy resin glues, and their practical application in providing shear connections between rectangular pretensioned beams. (see fig. 3) The problem of achieving uniform deflections has proved most difficult to solve because of the unequal transverse distribution of loads. Any number of keying arrangements has been tried, as well as transverse tensioning rods: all with some degree of success, but none could compare with the condition gained by a satisfactory method of gluing these members together, and thereby providing a homogeneous deck with all component members acting as one.

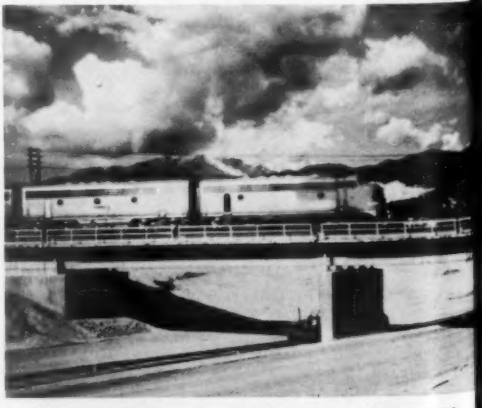


Fig. 2: 70 ft. spans of Santa Fe railroad bridge near Colorado Springs.

The use of epoxy resin glues along this line, as well as other uses, is presently being studied by the research group of the Association of American Railroads.

In line with general experimental work and test installations about our country, a short discussion to demonstrate the various areas of potential use of prestressed concrete might be in order.

Probably the greatest volume of activity at this time is in the field of railroad track ties. The American railroads have followed the European installations of precast, plain, post-

g. 3: Epoxy resin used as a shear connection between restressed beams. A small copper steam line was aced in the joint and steam was applied to furnish at, which set up the epoxy grout in about 15 minutes.

tensioned, and pretensioned concrete track ties with considerable interest. The laboratory, of Association of American Railroads, has conducted fatigue and other structural tests on concrete ties of various designs. A great deal has been learned concerning the design of a satisfactory prestressed concrete tie, not only for its physical dimensions and construction, but also a method of fastening the rail to the ties and satisfactory insulation of the assembly for use in signalled or coded track circuits. (fig. 4)

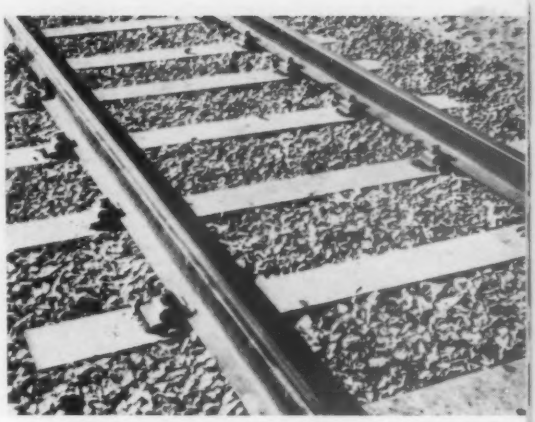


Fig. 4: A section of main line track constructed with prestressed concrete ties on the Atlantic Coast Line. Tie spacing was increased from the normal 20 in. to 30 in.

At the present time the Atlantic Coast Line and Seaboard Railroads each have test installations of concrete ties of approximately one-quarter of a mile in length. The ties used in these experiments are those designed by the research group of the laboratory of the Association of American Railroads in Chicago. (see fig. 5) Another small test section of prestressed concrete ties, based on European designs, is under field test at San Francisco, California, by the Western Pacific Railroad Company. (see fig. 6 and 7) These in-

Fig. 5: A typical rail to prestressed tie connection.

Fig. 6: Test section of prestressed ties on the Western Pacific Railroad.

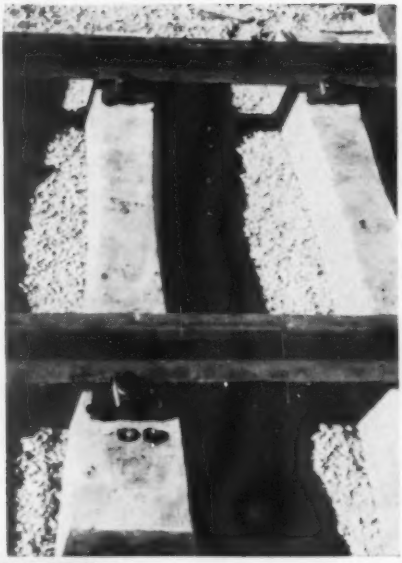


Fig. 7: Connection used on the Western Pacific section.

Fig. 8: Prestressed concrete track crossing.

stallations are being watched very closely, and tie manufacturers are studying methods of production wherein it is hoped that through standardization and use, quantity production can achieve a low-priced concrete tie competitive with timber. To date, each concrete tie, with necessary fastenings, costs approximately $11.00 in place. The concrete tie, without fastenings, costs from $8.00 to $9.00, for which we expect this tie to give us a service life of 50 years or more. With a hoped-for reduction in production costs, a possible slight change in tie spacing, and considerable extension of product life in track, prestressed concrete ties should become most attractive to railroad engineers responsible for economical track construction and maintenance. The American railroads have replaced an average of 24 million timber ties a year over the last ten years, so it is most evident that this potential is certainly worth considering.

So far as railroad bridge construction is concerned, the most popular use of prestressed concrete has been the short-span ballast rock type structure, although I am positive this use will expand to much longer spans in the future. In checking the number of timber trestles in this country, I have relied on information received from our Association publications, and found that there are a total of about 1800 miles of timber structure in use today on American railroads. At present the average service life of a creosoted timber ballast deck trestle is approximately 38 years. It was reported that nation-wide we are replacing or renewing some 170,000 track feet of timber trestle each year. Therefore, the magnitude of this potential, for the use of prestressed concrete in this one category, is staggering.

The prestressed concrete ballast deck trestle will fill the needs of the railroads in several ways. First, the expected service life should be considerably greater than that of timber; likewise, the maintenance of such a structure, during its entire life, should be considerably less. Although in fairness to both materials, I might add at this point that extended service life of a bridge structure is not always a criterion of economy, since in a great many cases the possibility of future track realignment, or changes, together with structure eliminations, place the shorter life of the timber more practical. If however the reuse in precast-prestressed segments can be developed it may alter this situation.

Another important feature of prestressed concrete, or concrete in general, is the absence of that old *hazard—fire*. As the chief engineer of a railroad, with only one main artery, I can assure you that this feature is of utmost importance. Cut the artery and you are definitely *"out of business"*.

Several railroads have invested in prestressed concrete structures, and the costs, although slightly higher at this time than treated timber, will, I am certain, be considerably lower in the future as more experience in fabrication and erection is gained. Out of 40 some-odd railroads questioned, six had experience with prestressed concrete bridges.

The Western Pacific has constructed with the exception of caps, a completely prestressed concrete trestle on the West Coast. (see fig. 12) A second structure is soon to be under construction and the members are now being fabricated; and a third bridge is coming off the design board. The cost at present will prob-

VOLUME TIGHTLY BOUND

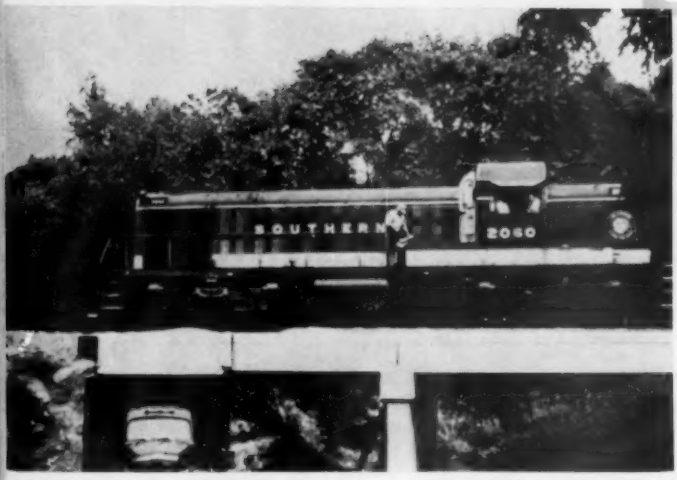


Fig. 9: Bridge on the Southern Railroad near Dry Forks, Va.

Fig. 10: 5 span prestressed bridge in Florida. This bridge was subjected to various dynamic tests and performed very well.

Fig. 11: 36 span prestressed bridge for the Atlantic Coast Line.

Fig. 12: 4 span bridge built under traffic for the Western Pacific R. R.

ably run about $350.00 to $375.00 per track foot, built under traffic. I have just taken bids on the second prestressed concrete trestle mentioned above, however it is not to be constructed under traffic conditions and I am certain that the cost will not exceed $300.00 per foot. Were I successful in using a precast cap I am certain the cost would be less. If trestles were of sufficient length to permit assembly line fabrication by the segment methods, or of standard segments, the costs would be considerably lower than as stated. Also, West Coast labor has a deterrent effect on costs. As an example: a large double-track, prestressed railroad structure was constructed in the southern section of our country simultaneously with the one built by the Western Pacific.

Fig. 13: C. & O. retarder yard tower at Newport News, Va.

Fig. 14: Yard tower for the Western Pacific R. R.

Their costs were over $100.00 less a track foot because of two facts, quantity and availability of labor.

Our limited experience with these type structures makes the figures indecisive at this time. It appears, if based on only a 50-years' life expectancy of a structure and not considering the fire hazard potential, that timber is still the most economical. It is felt with the increasing use of prestressed concrete and the subsequent standardization of structural sections, coupled with the increasing difficulty in securing suitable select timbers, that the picture

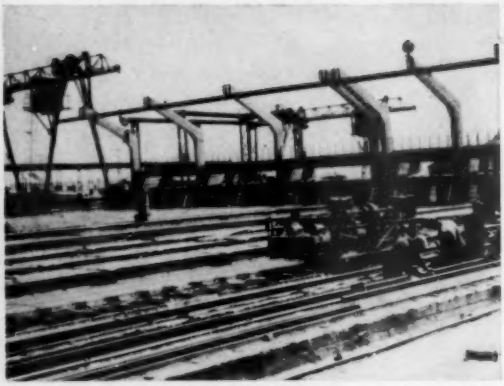


Fig. 15: Layout of a prestressed concrete tie plant in Baku, Russia.

will change in the immediate future.

The use of prestressed concrete in building construction, by the railroads, has been much the same as in all other industries. Generally all new structures are subject to analysis for the most economical form of construction before a design is prepared. This is based on a good deal of study as to the use of the building, its expected life, cost, and permanency. Examples of several most interesting buildings as developed by railroads, in utilizing prestressed concrete, are two yard-tower structures. One is located at Newport, West Virginia, constructed by the C. & O. Railway, and the other by the Western Pacific Railroad Company in Stockton, California.

The C. & O.'s railway retarder tower utilized prestressed concrete throughout its supporting structures in the form of pylons, presenting a most interesting architectural design. (fig. 13)

The Western Pacific structure utilizes prestressed concrete in the tower legs supporting a precast concrete and steel superstructure, the legs are prestressed piles driven to an exact location and cut off at a proper elevation for the tower to rest upon. (fig. 14)

Large warehouse structures of all types are being built by railroads to service their customers. In this construction we can utilize all varieties of precast, prestressed concrete shapes which have become quite popular in this country over the past few years. Shops, tunnels, railroad crossings, dock facilities, snowsheds, all have benefited by the use of prestressed concrete.

Prestressing in Russia

When I first contemplated this subject, and the topic of this presentation, I had no idea that I would be fortunate enough to be a member of a team of American railway engineers, to visit Soviet Russia as a representative of the Association of American Railroads and our State Department. My experience and observations in Russia and Europe proper would take several hours to

Fig. 16: Tie forms, note hardwood dowels for connecting to rail by lag screws.

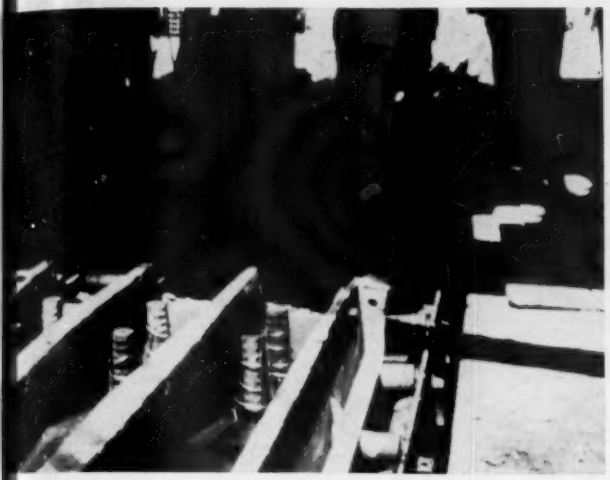


Fig. 17: Tie forms.

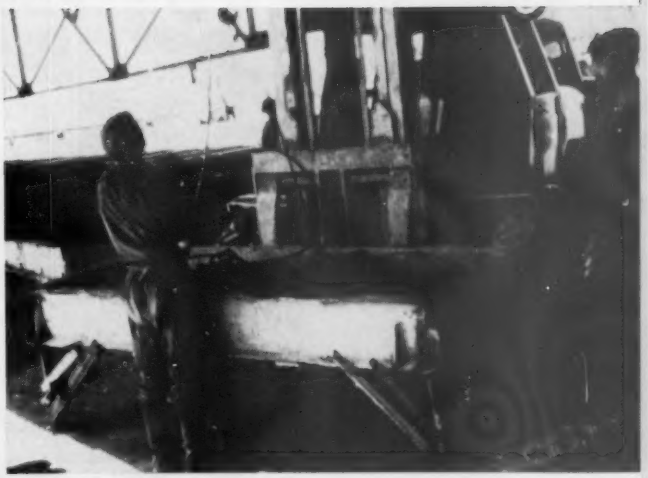


Fig. 18: Concrete placing equipment.

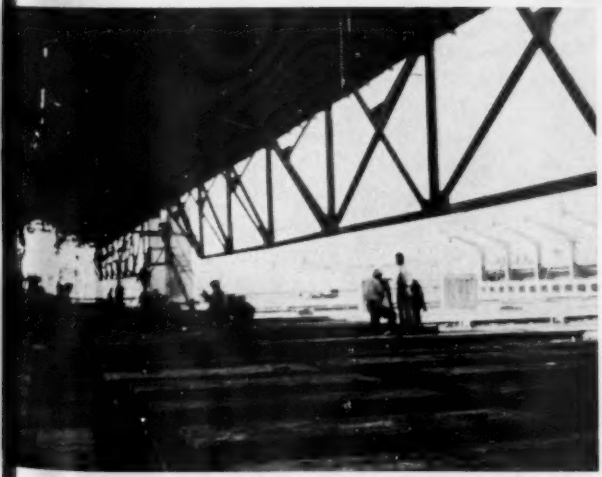


Fig. 19: Concrete placed in triple forms.

Fig. 20: Filled forms being slide down wires.

relate, but two points of special interest as concern prestressed concrete especially impressed me, and I am sure you would be interested as well.

At Baku, in the Province of Georgia, on the Caspian Sea, the Russians have located one of their ultramodern prestressed concrete crosstie and general structural segment fabricating plants. (fig. 15) At present they are manufacturing only prestressed concrete ties in a manner quite different from any method familiar to me. A triple tie steel form with 44 strands of 3 mm. crimped wires 81 m. long, (28 tie forms long), is placed under the concrete hopper.

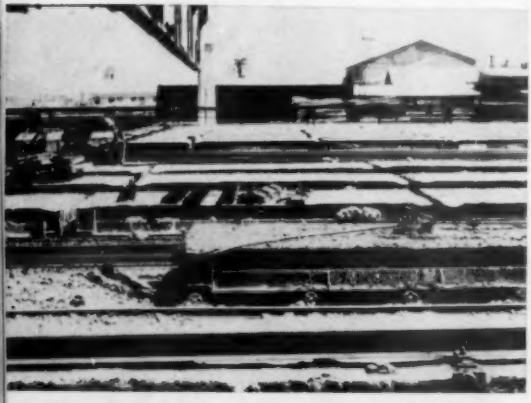


Fig. 21: Closeup of form being moved along wires.

These strands are threaded through the 28 forms into bulkheads at each end and stressed to 18,000 kilos per square centimeter or approximately one ton per wire. A rather moist but quick-setting concrete is then placed in the prepared form and vibrated. Following this action, the unusual portion of the operation begins. The triple tie steel form of vibrated concrete is now moved along the 44 prestressed wires to the end of each precasting line about 19 form lengths, the maximum distances from center, and revibrated. The next tie form is placed under the concrete hopper and a like action repeated,

Fig. 22

although moving one tie length lesser distance, such moves continuing until the center or pouring station is reached; this is repeated in the opposite direction, continuing for a full tie pour on each of the 36 identical lines. This process was planned to eliminate the necessity for moving the concrete placement equipment to each individual form. It would appear that there is considerable merit in this system so long as no decrease in bond or segregation of aggregate is experienced in the accomplishment of such a move. However through field observation I

Fig. 23

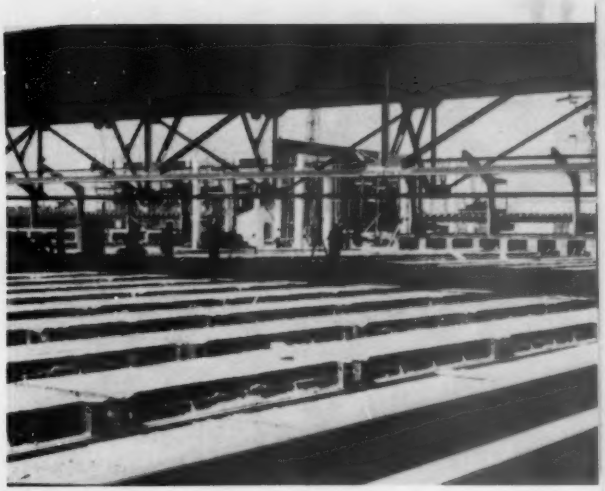
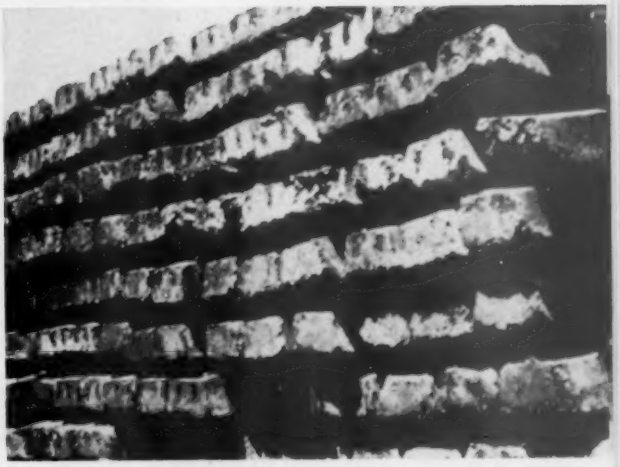


Fig. 24: Stressing end of the beds.

Fig. 25: Stack of finished concrete ties, contains ties of 3 gradiations.



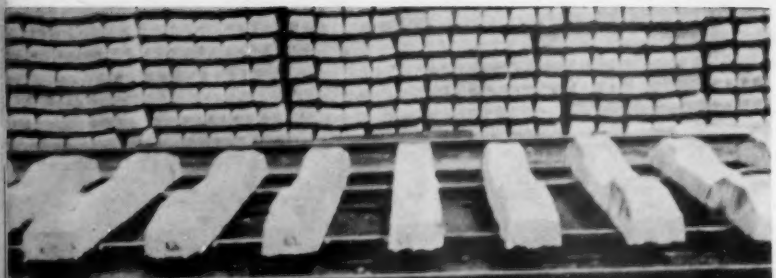


Fig. 26: Post-tensioned ties.

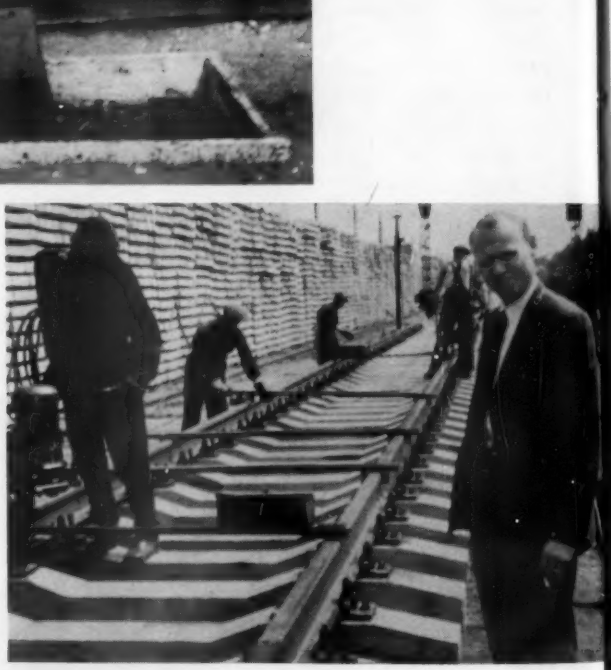


Fig. 27: A track section of post-tensioned ties.

Fig. 28: Track section between Stalingrad and Moscow using post-tensioned ties.

am afraid a considerable loss of bond does occur in movement of this concrete mass along the crimped wires. As far as I could ascertain, no test as to loss of bond, has been made. I inquired as to such a test but there was no information available.

In Moscow the construction of precast, prestressed concrete segmented unit-cell type apartment buildings is being developed by one of the government concrete plants at a most rapid pace. (fig. 29, 30, 31) Exterior and interior walls, floors, and roof slabs for these multi-story cell-like structures are field assembled through a stack on field arrangement

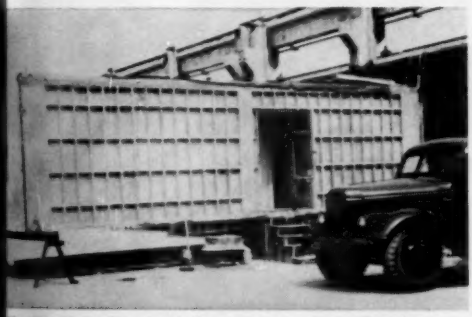


Fig. 29: Complete modular precast apartment unit.

plan. Here the method of pretensioning the concrete segments was most novel to us. Reinforcing wires were placed in the forms, an electric current was passed through the reinforcing, thereby heating the wires through electric resistance to a predetermined temperature. Next, a high-early strength concrete of a very dry consistency is placed in the form. The concrete is then compressed after it has set, by the wires which try to contract due to normal cooling. A similar use is made of rod steel, which is resistant heated before being placed in the form, after which quick-set concrete of a like very dry consistency is placed, vibrated, and then allowed to set before stress transfer. According to the

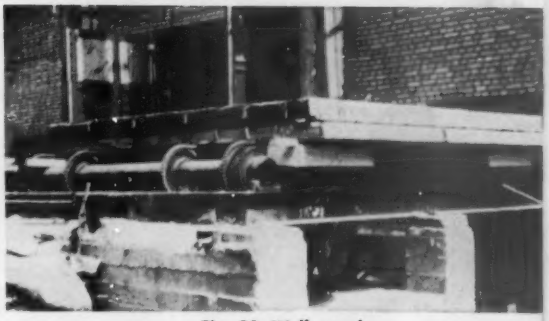


Fig. 30: Wall panel.

plant engineer, this method has proven highly successful, and the results obtained are comparable to standard prestressing procedures. Other similar but heavier sections are produced by this plant, including roof trusses of 6, 12, 18 and 24 m. lengths, similarly prestressed using reinforced steel rods. (fig. 32, 33, 34) The entire process is most unique, the greater part of the production being a continuous mechanical production-line operation.

In conclusion I would like to state that the future of prestressed concrete rests with its manufacturers and engineers. The results of the many tests and years of research have demonstrated its safety. Only you can demonstrate its economy by the use of your ingenuity to create, develop, and manufacture products which are competitive with their counterpart materials now in use.

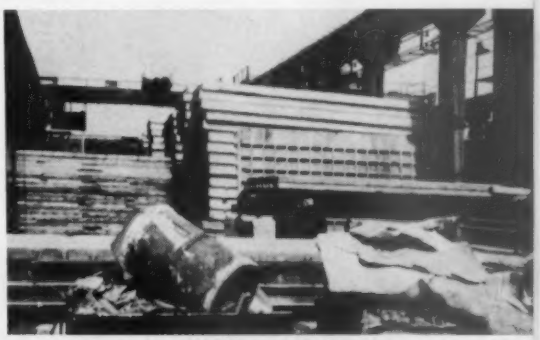


Fig. 31: Modular unit component parts.

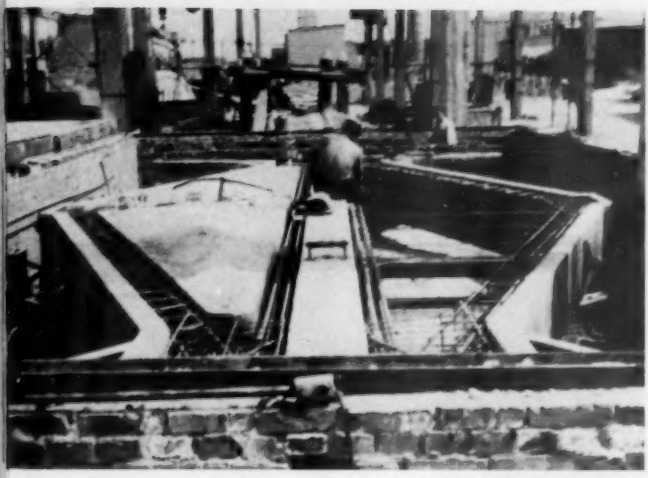


Fig. 32: Precast roof trusses, prestressed electrothermically.

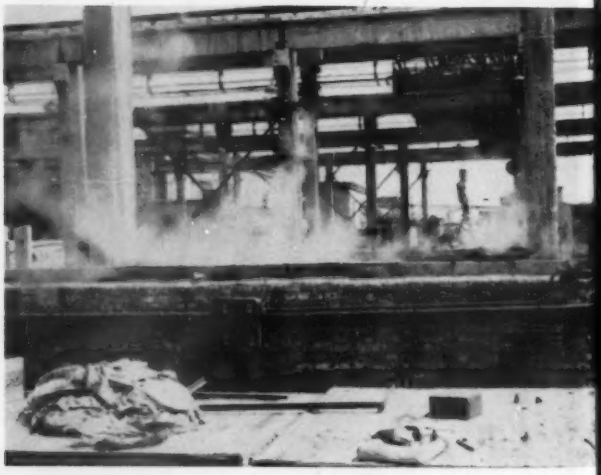


Fig. 33: Steam curing.

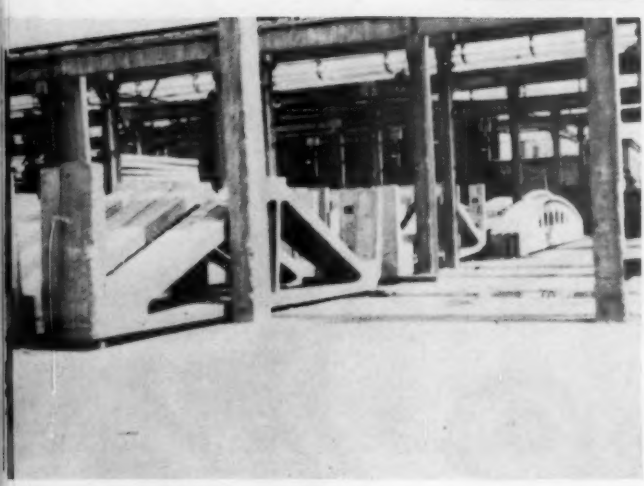


Fig. 34: Finished trusses.

Prestressing Solves Another Building Design Problem

by K. Yervant Terzian*

Presented at the Prestressed Concrete Institute Convention
New York City, September, 1960

Introduction:

In July 1959, the firm of Wallace and Warner was awarded the contract to design the U. S. Post Office Truck Terminal Annex at Philadelphia. The building, located between the existing Chestnut, Walnut and 30th Street viaducts, was ideally situated for expansion of the mail handling facilities of the main Post Office. At this site, the Annex is partially over the electrified tracks of the Pennsylvania Railroad which converge at the 30th Street Station.

Criteria:

*Structural Engineer,
Urban Engineers, Inc.,
Philadelphia, Pennsylvania

An aerial view of a scale model in fig. 1 shows, in the rear, a large truck maneuvering area, 480 x 115 feet, accessible from three city street viaducts. In the foreground, the building extends on the railroad tracks with the first floor approximately at the city street level, the second floor 20 feet above and the roof 36 feet from the first floor. Since the first floor presented the greatest problems we shall center our discussion on it.

The view in figure 2 is taken from the level of the railroad tracks under the Walnut Street viaduct again showing the building extending over the railroad tracks with deep post-tensioned girders and precast con-

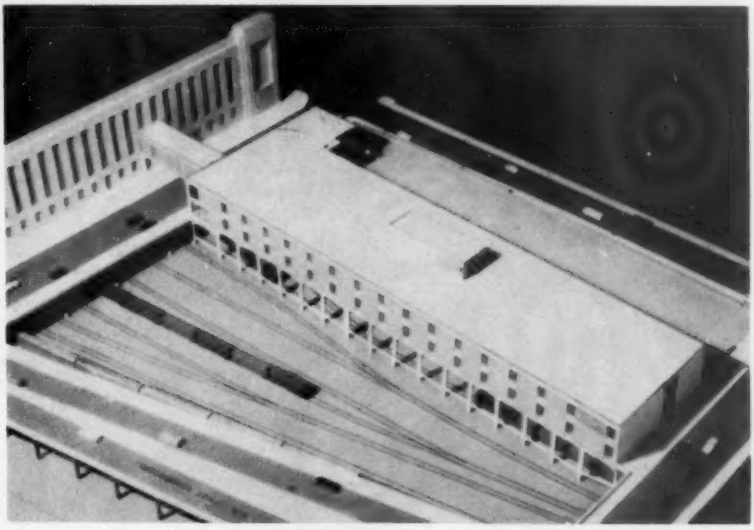


Fig. 1

Fig. 2

crete columns.

The 30th Street facade is pictured in fig. 3 with the mail trucks at different locations on the maneuvering area as well as at the lower level. The lower level of the 30th Street viaduct is a paved highway that will provide increased access for mail trucks to the building.

Figure 4 shows the existing site problems: The railroad tracks have to be maintained for traffic at all times as does the pattern of city streets with their heavy traffic. The shaded area is the truck maneuvering area designed for heavy AASHO H20-S16 loading, with 50 ft. square bays. The remaining portion of the structure is mainly over the railroad tracks. Girders carrying the loads to the foundation must support concentrated column loads from the floors above at their mid-span, and a floor live load of 100 lbs. per sq. ft. The bays measure 27.5 by 56 feet. Second floor live loads are either 200 or 100 lbs. per sq. ft. with second floor bays of 27.5 feet by 33.5 or 24.5 feet. Complicating the framing system are the numerous floor openings required for the mail handling conveyor system.

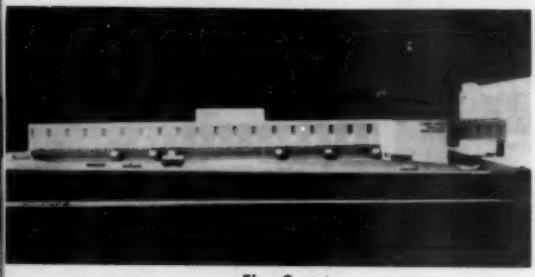


Fig. 3

We immediately decided that the truck maneuvering area was adaptable to the type of construction used for prestressed concrete highway superstructures. This meant box beams or I beams, which are sections that are competitively priced and readily available in the present market.

It was also apparent that standard highway box beams or I beams could not take the load of two story columns at their mid-span which was required in the area over the railroad tracks. The presence of the railroad tracks also makes it imperative to think in terms of the safety and continuity of operation of the trains and protective devices.

Fig. 4

In this regard, precast framing provides the minimum amount of interruption to railroad service. It also permits the elimination of safety platforms that may have been required for the fireproofing, and stripping of forms or placing of in-

sulation, for other types of construction materials.

In order to make it economically feasible, we felt that the framing should allow sufficient duplication of forms. This is not a problem for the maneuvering area because the main axes are at right angles and though the column spacing is not uniform, there are forty-five stringers of Type IV AASHO section and seventy-seven stringers of Type II AASHO section. However, because of the irregular geometrics over the railroad tracks, the framing plan was anything but simple. Certain bays abutting the streets had multiple floor levels and some very short spans abutted long spans all adding to the difficulty. We developed a uniform post-tensioned girder section for all 15 girders framing to the flared precast columns. The remaining girders were adapted from the same post-tensioned girder section combined with the vertical forms used for the box beams.

Clearances in general did not present a problem, but the exceptions seemed to occur at the wrong time and at the wrong place. The conveyor layout developed by the Post Office Department had to be altered at times for the sake of economical construction. On the other hand, it would have been practically impossible to erect safety platforms over the railroad tracks because of the tight clearances present under some of the conveyors.

The building code of the city of Philadelphia required that the building have a Class 1 fire rating which called for 2 in. of cover for the beams and columns, and ¾ in. for floor panels. Prestressed concrete has an advantage in this regard because it is fireproof without any additional treatment.

Design Features:

The maneuvering area framing consists of Type IV AASHO pretensioned stringers spaced at 11 feet on centers, spanning 58 feet, with a 9 in. reinforced concrete lightweight slab, supporting H20-S16 trucks in any direction. This loading is approximately twice the live load of the usual highway stringer which is parallel to the direction of traffic. Figure 5, shows the Walnut Street viaduct in the foreground and 30th Street lower level on the left side providing truck access to the building under the maneuvering area on the right side.

The stringers rest on neoprene

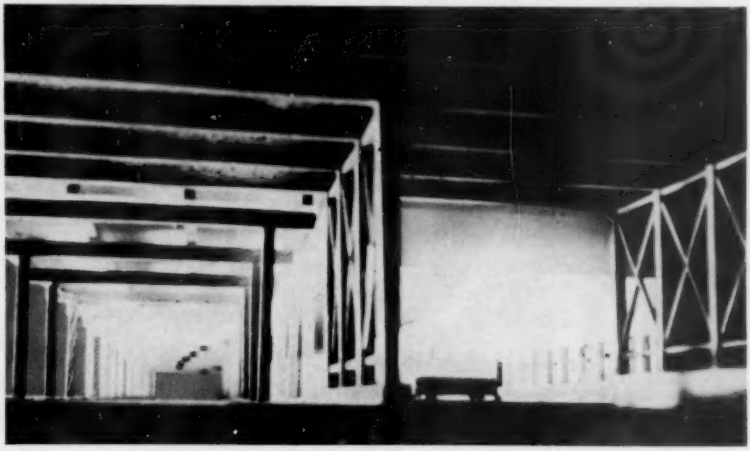


Fig. 5

pads that are set on cast-in-place concrete columns spaced at 11 feet at one end and on prestressed box beams along the 30th Street side at the other end. The box beams were necessary because the columns are spaced at 37 to 53 feet along 30th Street. The stringers are anchored in a reinforced concrete mass, 3 feet by 4½ feet that is designed to act compositely with the box beam. At the other end, an end diaphragm 2 feet by 3 feet ties the Type IV and Type II stringers and provides for continuity, but also allows lateral movement with regard to the columns.

Next to the main bay of 58 feet are two bays of 27 and 24 feet respectively framed with Type II AASHO pretensioned stringers spaced at 11 feet on centers with a 4 in. concrete wearing course and a 9 in. reinforced concrete slab with 3 ply membrane waterproofing. The provisions for movement, continuity and stability are similar to those of the previous bay. Particular attention has been given to precast concrete contact surfaces with the main requirement being distribution of high edge pressures to larger areas. This should prevent the cracking and spalling of concrete under the stringers.

The interior area consists of post-tensioned girders 8.5 feet deep spaced at 27.5 feet on centers spanning 56 ft., with 8 in. pretensioned floor panels and 2 in. concrete topping. (see fig. 6) This girder must support a column at mid-span plus a floor live load of 100 lbs. per sq. ft. The 8 in. pretensioned floor panels are standard items available on the market and competitively priced. They will be brought from the plant and erected during slack periods of train movement. Prior to erection, 2 in. fiberglass boards will be fastened mechanically to the underside of the panels to provide necessary insulation. These panels rest on unvulcanized rubber and cotton bearing pads ¼ in. thick by 3 in. wide. To prevent high stresses at the edges. In addition to the 2 in. concrete topping the panels are anchored all around by poured-in-place concrete that is in composite action with the girders.

The post-tensioned girder weighs

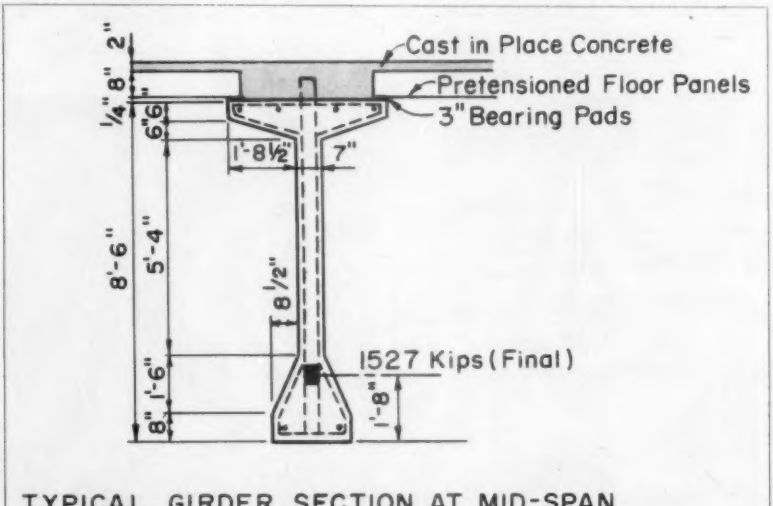


Fig. 6

40 tons and the concentrated column load near the center is 332,000 lbs., the live load for the floor panels, insulation, and 2 in. concrete topping is 2,750 lbs. per foot. The required prestressing force of 1477.3 kips after all losses with an eccentricity of 34.69 in. at mid-span produces a maximum compressive stress at the bottom flange of 2357 p.s.i. and a maximum tensile stress at the top flange of 31 p.s.i., with all superimposed loads, the maximum compressive stress is at the top flange and is 1115 p.s.i.

Under working loads shear stresses were computed at the concentrated column, quarter points and 10 feet from supports. The maximum shearing stress under working load was 170 p.s.i. Web reinforcement, designed by ultimate load, was provided by two legged #5 bars spaced from 6 to 9½ inches apart, which also extended above the top flange to act as shear connectors in composite action with the poured-in-place concrete. Ultimate strength of this girder was computed to be 2.55 (dead plus live).

The post-tensioned girders are detailed for the final prestressing force and eccentricity only at critical locations; thereby, giving the contractor the choice of wire, stranded wire, or bars, to use; as well as determining his own draping details. This provides the contractor the opportunity to present the most competitive bid. These girders will also be erected during slack periods of train movement and will rest on neoprene bearing pads on the precast reinforced concrete columns.

In order to provide for stability of construction, post-tensioned connections were studied between the columns and the girders. The design of a moment connection did not prove economical because of the relatively large disparity in moment of inertia of the column and girder; therefore, it was decided to treat it as a hinged connection. This hinged connection provides stability without incurring a substantial cost increase. It was also anticipated that post-tensioning would require small platforms for 2 workers at each column.

In addition to the transverse floor panels, precast tie beams are provided for lateral stability.

These tie beams are placed near the concentrated column loads and at the column lines. The tie beams at mid-span are seated next to a girder web stiffener and are post-tensioned with it. Tie beams at the column lines are seated and post-tensioned with the column connection.

It is important to note that all beams are seated and temporary scaffolding for erection is not required. Accordingly, erection costs should be lower.

Concrete for the precast beams, pretensioned I beams and floor panels is required to have a minimum 28 day strength of 5000 p.s.i.; and concrete for the post-tensioned beams is required to have a minimum 28 day strength of 6000 p.s.i. The poured-in-place concrete slab in the maneuvering area is required to have 3000 p.s.i. concrete strength with air-entraining additive and the 2 in. concrete topping on the floor panels is required to have 4000 p.s.i. concrete strength.

Forms for prestressed members shall be metal lined. This requirement will preclude the chance of a poor result without handicapping the construction cost because there is sufficient reuse. For instance, 45 Type IV A.A.S.H.O. beams, 77 Type II A.A.S.H.O. and 24 post-tensioned I beams are required. Box beam forms

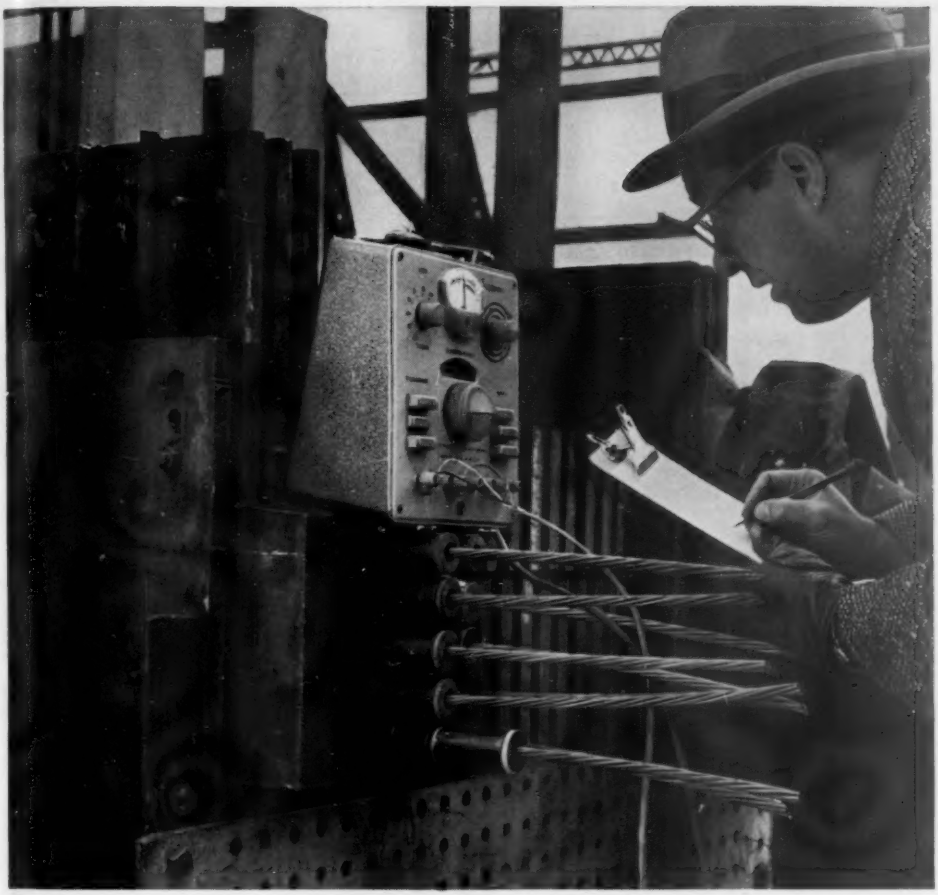
will be straight-forward and can be adapted from standard box sections or salvaged for future use.

Between the interior area of prestressed concrete and the maneuvering area is a bay of poured-in-place reinforced concrete construction. The reasons for this are threefold:

1. This bay has 2 lines of dump hole openings 2 ft-6 in. x 4 ft-2 in. spaced every 5.5 feet for a distance of 440 feet and terminates as a loading platform on one side which is 4 feet above the maneuvering area. In addition, there are two platform scale pits. It would be very impractical to precast this type of a bay.
2. This bay can serve as the rigid anchor for the precast construction which is very important for stability considerations in multistory buildings.
3. Stability of construction was obtained in the following manner:
   a. Adequate anchorage of beams to columns.
   b. Adequate bracing of main girders.
   c. Construction of a center bay of poured-in-place reinforced concrete.

Conclusion:

Prefabrication always presents particular problems for each building site, and the advantage of its application depends largely on the solution of all these problems; the advantage obviously increasing with size of the building site. Prefabrication requires great care in preparation of the constituent elements and calls for the use of proper equipment for handling. On the other hand, it leads to economy in scaffolding and formwork and allows greater speed of construction. Prestressed concrete solved the design problems for this building by proving itself adaptable to the many requirements.



How Bethlehem Engineers Aid Prestressed Concrete Fabricators

Friction-loss test conducted to meet Pennsylvania Highway Department specs

Meet Jim Scott, a Bethlehem research engineer. He is recording the micro-inch differential of Bethlehem stress-relieved strand under load at the Dunmore, Pa. plant of Concrete Engineering, Inc.

Jim is there because of a stringent test requirement of the Pennsylvania State Highway Department. To qualify as a supplier of deflected-strand prestressed members for highway bridges, a fabricator must furnish evidence that the frictional loss in his strand, from one abutment of his casting bed to the other, is no greater than 5 per cent. A measurement within this tolerance is accepted as proof that the prestresser's method of operation is satisfactory.

Under the watchful eyes of State Highway Department witnesses, Bethlehem conducted a 4-station load cell test. Tension links were used on 2 slopes and compression links at both ends of the 345-ft bed. Test results showed frictional losses of less than 3 per cent, well within the state requirements.

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Readers' Comment

DISCUSSION

TENTATIVE RECOMMENDED PRACTICES FOR GROUTING POST-TENSIONED CONCRETE*

By D. O. McCall
Vice President, Basalt Rock Company, Inc.

Gentlemen:

We would like to submit the following discussion regarding the proposed grouting practices.

Paragraph 103.4-As presently worded, the inclusion of aluminum powder is virtually mandatory, especially when considered in conjunction with Section 108.1. We question whether there is sufficient service record to even allow aluminum powder, let alone encourage it. Some years ago it was considered by a local governmental agency, and after considerable research abandoned for lack of substantiation. If such information is available, we would be most anxious to see it. In some members it is conceivable that 10% expansion could cause splitting. There is also cause for concern over electrolytic action.

Paragraph 105.2-We have done a great deal of grouting with a pressure pot and have had far better results with it than with the positive displacement pump. We have cut open many ducts that have been grouted with a pressure pot and found absolutely no air voids. Possibly one reason for our success with the pressure pot is that we have a very ample supply of high pressure air, which should possibly be a requirement if the pot is to be used. Most of the positive displacement

*June, 1960 Vol. 5, No. 2

pumps we have seen and/or used have been of not too large a capacity, which possibly conditions our opinion, but the fact remains we have had better luck with the pot than the pump and would think twice about bidding a specification that prohibited it. Probably in either case, the main point which should be stressed is that ample volume of flow be maintained.

Paragraph 107(d)-We have found the requirement of 100 psi injection pressure difficult to interpret and police from the inspection standpoint since it is impractical to actually measure the pressure of the grout in the duct. Most local agencies now specify "equipment capable of developing a pressure of 100 psi" which is more practical and achieves the intended purpose.

Paragraph 108.1-We would be interested to know the background of this requirement and also whether any grout has been used commercially which meets this requirement. In a mix with this much excess water some bleeding or water gain would be expected when tested in this manner unless some expansive agent is employed to offset it.

Certainly it is desirable to have a grout with no shrinkage, but we question the relationship between volume change measured from the time of placing to initial set, and the total volume change. In (a) shrinkage is not allowed while in the "plastic state". The recommended formula would measure volume change in the fluid state. If plastic change is desired, shouldn't it be measured from initial to final set? Also, how is the set to be determined? The usual procedure with a Gillmore needle will not work on this type of grout.

The committee is to be commended for their efforts, especially since so little is available on this subject.

Comments by the Grouting Committee on Mr. D. O. McCall's discussion of the "Tentative Recommended Practices for Grouting Post Tensioned Prestressed Concrete"

103.4 The use of aluminum powder, limited to that amount which will cause 10% expansion, was agreed by the majority of the committee to be extremely desirable. The use of aluminum powder is generally recognized as being desirable.

As to the question of service record, aluminum powder has been in expansive concrete for a good many years without harm to either the concrete or its reinforcing steel.

With respect to a 10% expansion causing splitting, a simple calculation of gas pressures indicate that this is very unlikely. Based upon a completely gas tight duct a gas volume which would cause a free expansion of 10% in a grout containing 4% (this is a normal amount) entrained and entrapped air, the maximum pressure which could develop would be 37 psi. It is very unlikely that:

1. The duct would be tight enough to hold the pressure over the period that the gas is forming.

2. This pressure, if developed, could exert enough force in the surrounding concrete to cause cracking.

105.2 Most recommendations and specifications for pressure grouting specify a pump. In some cases the use of the pressure pot is specifically prohibited. It is generally considered that the use of a pressure pot has been the cause of frequent poor grouting results. The main reasons for its disfavor are:

1. Running out of grout without the operators knowledge.

2. Difficulty in controlling the rate of injection.

3. Limited maximum pressure.

107 (d) If the grouting pump is equipped with a working gauge as it should be, very little difficulty in recording definite pressures will be encountered. Except for completely gas tight grout ducts, the maintenance of a minimum pressure for a minimum time will expel some trapped air and bleeding water. The amount expelled will depend upon grout pressure, time and the porosity of the duct. It is agreed that if the duct is completely gas tight then the holding of pressure can be of little value. This condition rarely exists. As the "Tentative Recommended Practices" only call for a minimum time of ten seconds, this should cause very little delay on the job. The benefits derived from expelling some of the entrapped air and bleeding water will far outweigh the cost of the extra time.

108.1 It is generally agreed that a reduction in volume (shrinkage) of grout is undesirable. Usually, the greatest amount of volume reduction will take place prior to final set. Some of this volume reduction will be bleeding. In normal concrete construction this volume change is of less importance as it is manifested in a vertical reduction in height prior to final set of the concrete. In the case of the grout in an enclosed duct any reduction in volume will leave a void. These voids usually form in two places: on the under side of the stressing units and the upper area of the duct. These voids can be harmful if:

1. Under freezing conditions they are water filled and expansion cause splitting.

2. Reduction of bond is great enough to be detrimental.

3. Other factors could produce an electrical potential for electrolyte action.

As the use of an expansive material will greatly overcome this reduction in volume, it is generally considered to be desirable.

The committee thought it was proper to specify a simple method of determining this plastic volume change. The method shown in the "Tentative Recommended Practices" has been used and is found to be a simple and inexpensive job control.

To answer the more specific questions in Mr. McCall's discussion:

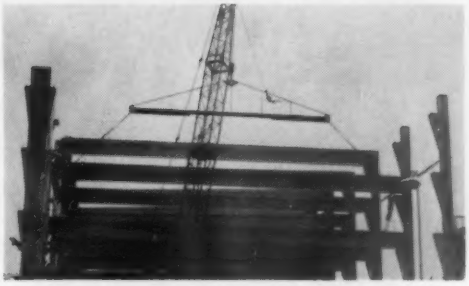
1. There is on the market one or more grouting "aids" which will meet the requirements of the "Tentative Recommended Practices." These "aids," we believe, all contain aluminum as the expansive agent.

2. Whether the "plastic" state is proper or "fluid" state would be better is, we believe, only a choice of words where it is used in *paragraph 108.1 (a)*. The thought of the committee was that a material placed in a container with an open top would only move up or down during expansion or contraction while it is in a "plastic" state.

3. Somewhere between the initial and final set the grout will no longer be in a "plastic" state and contraction will be in all dimensions. *Paragraph 108.1 (d)* specifies that the grout column "Hs" is to be measured after initial set. It is not critical that the initial or final set of the grout need be determined.

It was the intent of the Technical Activities Committee of the PCI that the "Tentative Recommended Practices" be what the name implies, only *tentative*. As additional experience dictates changes, these should be incorporated.

PRESTRESSED CONCRETE PARKING GARAGE SAVES CITY OF BEVERLY HILLS $186,000



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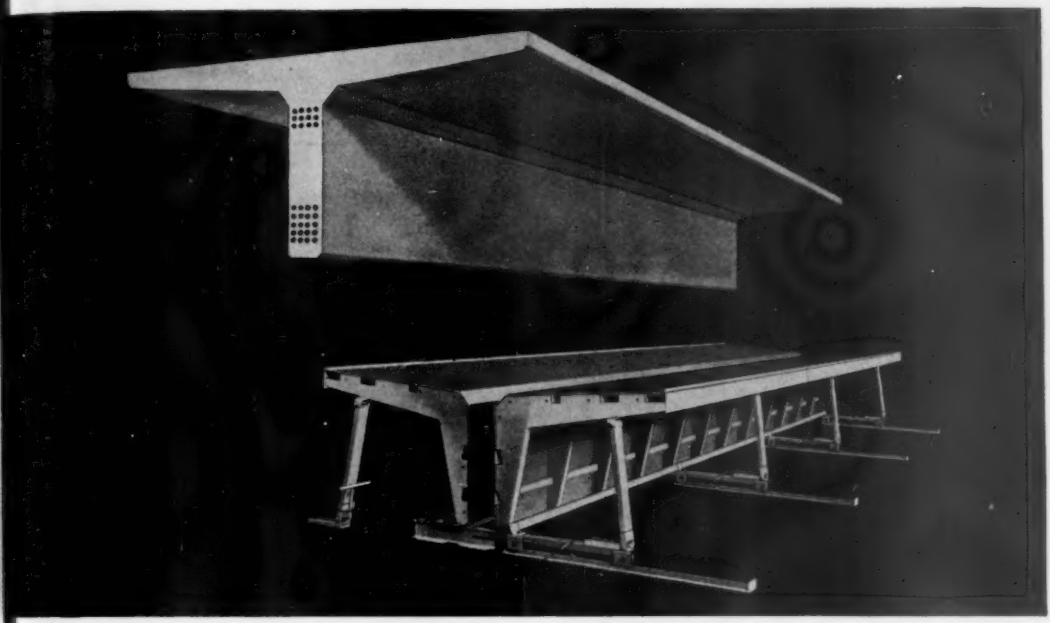

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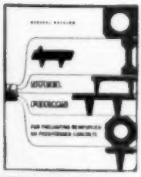

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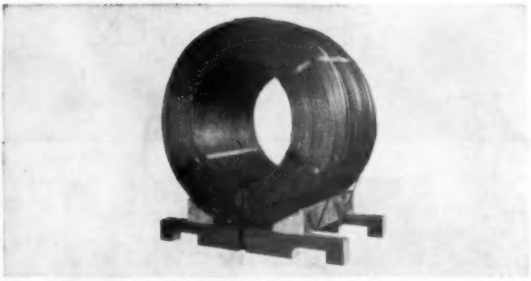

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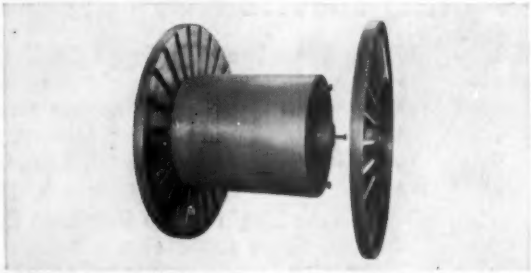

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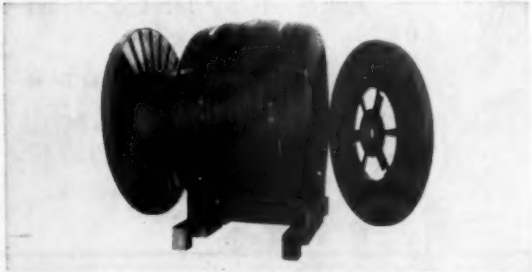

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